THE ASSOCIATION OF ENGINEERING
AND SHIPBUILDING DRAUGHTSMEN

AESD

DESIGN OF OVERHEAD TRAVELLING CRANE STRUCTURES

Session 1958-59

By

E. C. W. SWANN
A.M.I.Struct.E.

Price 3/6

THE ASSOCIATION OF ENGINEERING
AND SHIPBUILDING DRAUGHTSMEN

Design of Overhead Travelling Crane Structures

by

E. C. W. SWANN, A.M.I.Struct.E.

Published by The Association of Engineering and Shipbuilding Draughtsmen, Onslow Hall, Little Green, Richmond, Surrey

SESSION 1958-59 6/59

Design of Overhead Travelling Crane Structures.

By E. C. W. Swann, A.M.I.Struct.E.

An overhead travelling crane structure is a complex frame subject to many loads, some of which are dynamic or shock loads.

The normal type of crane consists of two main girders, which support the crab or lifting unit. Either one or both of the main girders are braced to an outrigger or auxiliary girder to give both lateral stability and support to the long travel gear, platforms and electrical equipment. The girders are bolted to end carriages; these house the main travel runners on which the crane travels on the gantry.

Figs. 1 and 2 show the general outline of crane, naming the main components.

In recent years several technical investigations have been carried out on crane structures, notably those investigated by the teams at Leeds University and the British Iron and Steel Research Association.

The results of these investigations have been found most useful in confirming the behaviour of crane structures and assessing the magnitude of various dynamic loadings.

There are two main specifications dealing with overhead crane girder structures in this country.

1. British Standard Specification 2573 (1955).
2. B.I.S.R.A. Crane Specification.

The B.S. 2573 covers all classes of cranes and the designs that follow are based on this specification.

The B.I.S.R.A. specification deals solely with heavy duty cranes as used in iron and steel works.

The method of calculations given in the examples that follow have been devised over a number of years in order to obtain good accurate results by practical methods.

Class of Crane.

Electric overhead travelling cranes are divided into five classes dependent on the number of hours of service per year and the duty performed.

Class 0. Hand and light powered cranes used for up to 1,000 hours per year.

Class 1. Medium duty industrial cranes for intermittent use in stores or light machine shops.
The hours of duty are between 1,000 and 2,000 hours per year.

Class 2. General duty cranes as used in workshops, warehouses, non-ferrous foundries, railway goods yards.
The hours of duty are between 2,000 and 3,000 hours per year.

Class 3. Steelworks service and light process cranes, heavy duty foundry work, light magnet and grabbing duty.
The hours of duty are between 3,000 hours and 4,000 hours per year.

Class 4. Continuous process cranes for steelworks; continuous magnet duty and continuous grabbing duty.
The hours of duty are over 4,000 hours per year.

Crane Loading.

The loads carried by a crane structure are as follows, and the design should cover any combination of these :—

a. Safe working load on hook.
b. Impact of safe working load.
c. Weight of crab.
d. Self weight of structure.
e. Platforms and walkways.
f. Long travel machinery.
g. Wind loads.
h. Inertia forces in horizontal direction.
i. Torsional loads from equipment cantilevered from girder.
j. Torsional load from long travel motor.
k. Erection forces and effects.

Deflection.

Many values have been put forward by various bodies in the past and the value that the deflection due to the live load only should not exceed 1/1200 of span gives satisfactory results.

This value can easily be checked during normal testing.

If, however, the crane has a heavy special duty crab, sometimes weighing several times more than the safe working load, then it is advisable to include the weight of the crab into the deflection allowance. In this case the deflection due to the safe working load plus the weight of the crab should not exceed 1/900 of the span.

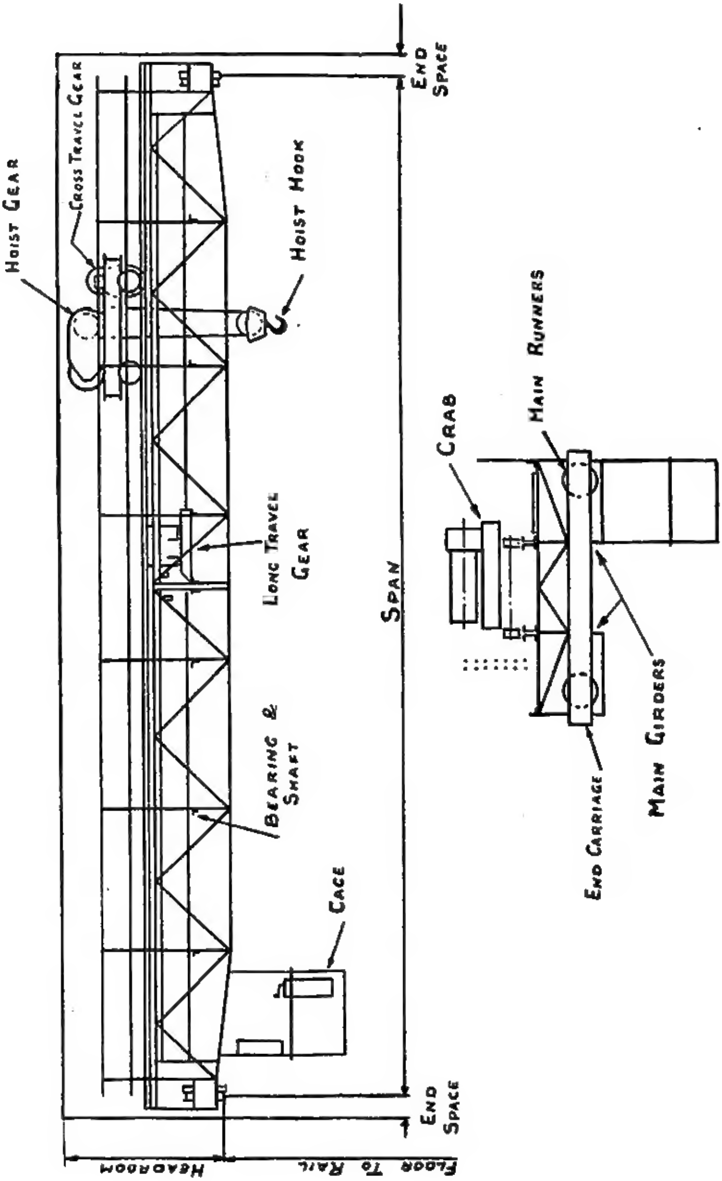

Hoist Gear
Cross Travel Gear
Hoist Hook
End Space
Long Travel Gear
Span
Bearing & Shaft
Cage
End Space
Headroom
Floor To Rail
Crab
Main Runners
End Carriage
Main Girders

Fig. 1.

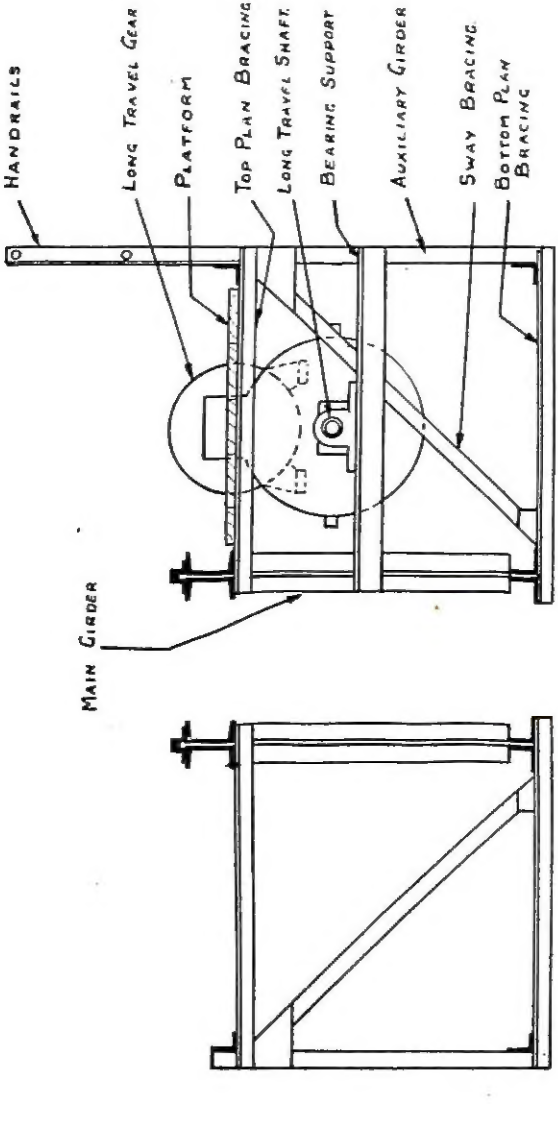

Handrails
Long Travel Gear
Platform
Top Plan Bracing
Long Travel Shaft
Bearing Support
Auxiliary Girder
Sway Bracing
Bottom Plan Bracing
Main Girder
Geared Girder
Ungeared Girder

Fig. 2.

In the R.S.J. type of girder the deflection should be very carefully watched if the depth to span ratio exceeds 25, as the girders can develop an unpleasant bounce on loading.

Depth of Girders.

The depth of crane girders is quite often decided by local site headroom clearances, but as a general rule, a satisfactory economical design is obtained if the following proportions are used.

Lattice Girders.	Effective depth = $\frac{1}{14}$ of span.
Plate Girders.	Depth to be not less than $\frac{1}{18}$ of span.
R.S.J. Girders.	Depth to be not less than $\frac{1}{25}$ of span.

These proportions are a good guide for cranes up to 25 ton capacity ; above this the depth of the girder should be increased slightly.

Width of Girders.

The span of an independent girder without outrigger bracing should not exceed 200 × the least radius of gyration.

The overall width of either a plate or lattice girder complete with outrigger girder is usually decided by the space required for walkways, long travel gear and electrical equipment, but the width should not be less than $\frac{1}{20}$ of span.

Rails.

There has been a lot of controversy in the past as to whether the rail section should be included in the flange area ; the latest trend is that, providing an allowance is made for wear and the rail fastenings are sufficient to withstand the horizontal shear, then the rail area can be taken as a useful section.

The positioning of joints then becomes an important feature in design and the control of this in a structure, which is often a "bought-out" commodity, is extremely difficult.

Therefore no laid down rules can be set in this matter, and it should be decided entirely on local conditions.

The following give a size of rail suitable for Class 2 cranes :—

32 lbs. per yard bridge rail :—	Wheel load	7·5 tons.
40 ,, ,, ,, ,, ,, :—	,, ,,	10·5 tons.
56 ,, ,, ,, ,, ,, :—	,, ,,	15·0 tons.
70 ,, ,, ,, ,, ,, :—	,, ,,	30·0 tons.

Types of Girders.

Fig. 3 shows the more usual types of girders used ; there are, of course, several variations to these, but mainly they fall into one of the groups shown.

a & *b*. These are the R.S.J. and compounded R.S.J. girders, and this type is used for cranes up to 30 ton capacity. The span usually does not exceed 50′-0″, dependent upon the crane capacity. A light outrigger is fixed to one girder to carry the long travel machinery and platform.

c & *d*. Welded or riveted fish bellied box girders, used as in *a* & *b* with a light outrigger. This type is used for all tonnages up to about 70′-0″ span. These girders are of less depth than the lattice girder and thus are very useful in buildings with restricted headrooms.

e. A single web plate girder of either welded or riveted construction; both the main girders should be braced to lattice auxiliary girders for stability.
The range of usefulness is similar to types *c* & *d* although spans of 80′-0″ can be made econonically in welded designs.

f. The box lattice girder, often used for slow moving cranes of capacities up to 40 ton and spans up to 70′-0″. A light outrigger is required on one girder to carry the long travel gear.

g. This is a lattice type girder commonly used to-day and results in an economical rigid structure. With variations to the boom sections this type of girder can be used for all tonnages up to 100′-0″ span.

h. Similar to type *g* but of welded construction; as in all welded lattice girders, great care needs to be exercised in the details of panel points joints to reduce the possibility of fatigue failure.

i. Several variations of this type of tunnel or through girder have been produced and they are most useful over 100′-0″ span.
It can also be used to suit particular site conditions, such as where the storage space below the crane girders needs to be a maximum.
The portal bracing above the crab should be of reasonable depth in order to keep the loads in the bracings to a minimum.

Camber.

It is usual to build lattice and plate girders with an initial camber of 1″ per 100′-0″ of span.

Impact Factors.

This factor is applied to the safe working load of the crane to cover for the various degrees of duty carried out by the crane, and allows for shock loading and inertia forces in the vertical direction.

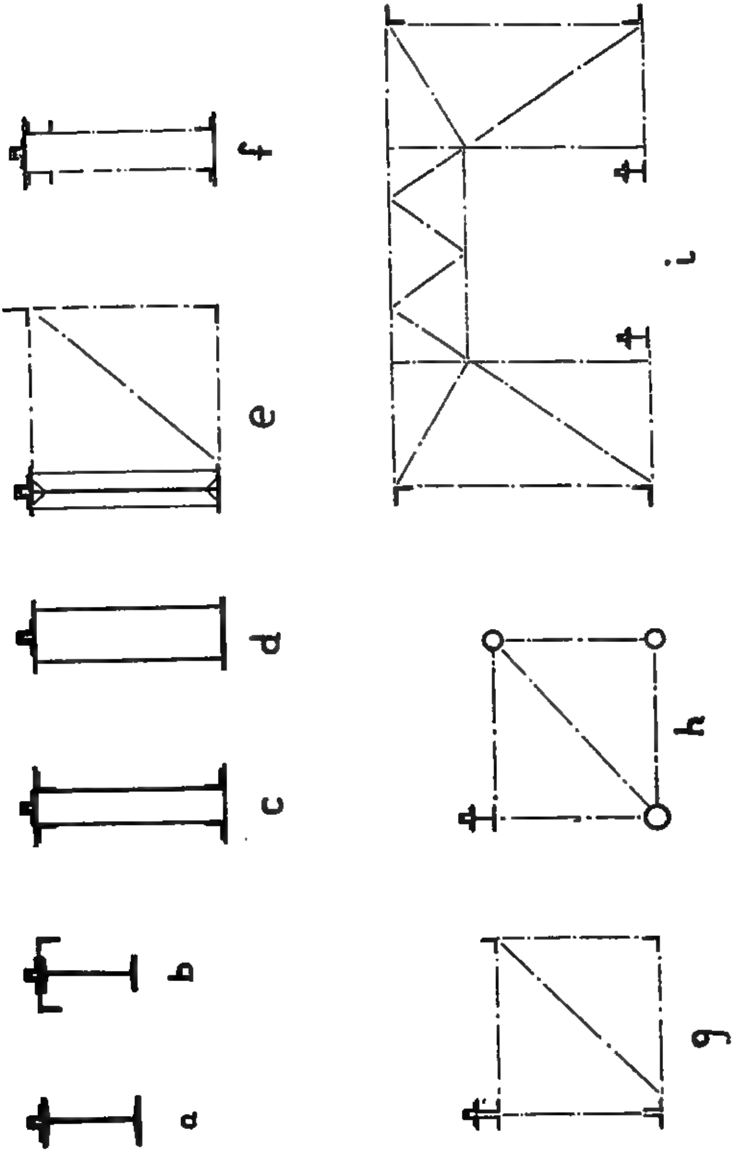


Fig. 3.

The value of the impact factor is as follows :—

Class 0	1·1
Class 1	1·2
Class 2	1·35
Class 3	1·5
Class 4	1·65

Lateral Inertia Forces or Longitudinal Surge.

This force is the horizontal load applied to the girder caused by the acceleration or retardation of the crane and acts at right angles to the crane girder, thus the term lateral inertia force. This force when transmitted to the gantry track is known as the longitudinal surge in gantry design. The recommended values of this force in use at the present time are :—

Class 1.	$\frac{1}{25}$ of vertical load.
Class 2.	$\frac{1}{20}$,, ,, ,,
Class 3.	$\frac{1}{15}$,, ,, ,,
Class 4.	$\frac{1}{10}$,, ,, ,,

or $\frac{V}{6000}$ whichever is the greatest.

Where V = speed of long travel motion of the crane in ft. per min.

This factor is applied to both the live and the dead loading.

Stress Factor.

The permissible working stresses are given by the product of a basic stress dependent on the material and a stress factor dependent on the class of crane. These factors are as follows :—

Class 0.	Stress factor 0·9
Class 1	,, ,, 0·85
Class 2.	,, ,, 0·80
Class 3.	,, ,, 0·75
Class 4	,, ,, 0·70

Basic Tensile Stress.

In B.S.S. 466 (1953) the specification for overhead electric cranes, the basic tensile stress for both axial and bending stresses was 6 tons per sq. in., but there was not an impact load or stress factor to be applied.

The introduction of B.S. 2573 (1955) brought in a new series of allowable basic stresses which generally are as follows for mild steel :—

Axial stress on net effective area	9·0 tons per sq. in.
Bending stress on extreme fibres of plate girders	9·5 „ „ „
Bending stress on extreme fibres of beams other than plate girders ...	10 „ „ „

To these must be applied the appropriate stress factor, and the loading with which they are used must be calculated with the necessary impact factor applied to the live loading.

For basic tensile stresses of other steels see Table 1, B.S. 2573 (1955).

Basic Compressive Stress.

Similarly, the compressive stresses have been increased from B.S. 466 (1953), and again the stress factors and impact factors must be included in the calculations.

The basic allowable stresses for mild steel are as follows :—

Axial stress ($l/k=0$)	9·0 tons per sq. in.
Bending stress on extreme fibres of plate girders	9·5 „ „ „
Bending stress on extreme fibres of beams other than plate girders ...	10·0 „ „ „

The axial stress for struts for varying effective slenderness ratios is given in Table 1 of the Appendix. The respective stress factors having been applied for the various classes of cranes.

The bending stress for beams is given in Clause 10, B.S. 2573 as follows :—

F_{bc} = $A + K_2 B$ tons per sq. in. or the appropriate bending stress given above, whichever is the least.

Where F_{bc} = basic bending stress.

Values A and B are given in Table 2 of Appendix in terms of l/k_y and d/t_e.

e = Effective length in inches.

k_y = Radius of gyration in inches of beam section perpendicular to plane of bending, at the point of maximum bending.

d = Overall depth of beam in inches at point of maximum bending.

t_e = Effective thickness of flange in inches = $K_1 \times$ mean thickness of the horizontal portion of the flange.

$$\text{Mean thickness} = \frac{\text{area of horizontal portion of flange}}{\text{width}}$$

K_1 Allows for the curtailment of the thickness or breadth of the flange and is given to various values of N.

$$N = \frac{\text{area of both flanges at point of minimum bending}}{\text{area of both flanges at point of maximum bending}}$$

N =	1·0	0·9	0·8	0·7	0·6	0·5	0·4	0·3	0·2	0·1	<0
K_1 =	1·0	1·0	1·0	0·9	0·8	0·7	0·6	0·5	0·4	0·3	0·2

K_2 Allows for inequality of tension and compression flanges and is given to various values of M.

$$M = \frac{\text{Moment of inertia of compression flange about YY axis}}{\text{Moment of inertia of whole section about YY axis}}$$

M =	0·5 or greater	0·4	0·3	0·2	0·1	0
K_2 =	0	−0·2	−0·4	−0·6	−0·8	−1·0

Combined Bending and Compression.

Members subject to combined bending and compressive stresses should be designed so that the quantity

$$\frac{f_a}{F_a} + \frac{f_{bc}}{F_{bc}} \text{ does not exceed unity}$$

where f_a = axial compressive stress.

F_a = allowable compressive stress.

f_{bc} = compressive stress due to bending.

F_{bc} = allowable compressive stress for bending.

Shear Stress in Web Plates.

The average shear stress on the effective area of the web should not exceed for mild steel.

For unstiffened webs 6·0 tons per sq. in.

For stiffened webs $6\left[1{\cdot}3 - \dfrac{b/t}{250\left[1 + \frac{1}{2}\left(\dfrac{b}{a}\right)^2\right]}\right]$

or 6·0 tons per sq. in., whichever is least.

Where a = the greater unsupported clear dimension of web in a panel.

b = the lesser unsupported clear dimension of web in a panel.

t = thickness of web.

Again these stresses should be used in conjunction with a stress factor and impact loading.

Shear and Bearing Stresses in Bolts.

The basic values for these and Class 2 values are given in Table 3 in the Appendix.

Slenderness Ratio.

The maximum permissible slenderness ratios of a member in compression are as follows :—

Main structural members	140
Secondary members and wind bracing	240
Flanges of beams	200

Effective Length of Compression Members.

The effective length of a compression member = l. The values of this are given below as a proportion of L (length of strut from centre to centre of intersections with supporting members).

1. Effectively held in position and restrained in direction at both ends $l = 0{\cdot}7\,L$
2. Effectively held in position at both ends and restrained in direction at one end $l = 0{\cdot}85\,L$
3. Effectively held in position at both ends but not restrained in direction $l = 1{\cdot}0\,L$
4. Effectively held in position and restrained in direction at one end and at the other partially restrained in direction but not held in position $l = 1{\cdot}5\,L$
5. Effectively held in position and restrained in direction at one end but not held in position or restrained in direction at the other end $l = 2{\cdot}0\,L$

Thickness of Members.

Cranes in clean atmospheres and indoors, such as workshops, warehouses, etc., a minimum thickness of $\frac{5}{16}''$ for main structural members and $\frac{1}{4}''$ for secondary members is advisable.

If the crane is outdoors all members should be $\frac{5}{16}''$ thick, whilst cranes in corrosive atmospheres should have special consideration for each case.

Design of Lattice Crane Girders.

Data.

Outline of crane girder	Fig. 5
Class of crane	2
Safe working load	15 tons
Span	60′-0″

Impact factor	1·35
Weight of crab	4·0 tons
,, ,, one main girder	3·5 tons
,, ,, one auxiliary girder	0·9 tons
,, ,, one plan bracing	0·5 tons
,, ,, shafting and bearings	1·0 tons
,, ,, long travel motor	0·7 tons
,, ,, platform	0·6 tons
,, ,, handrails	0·05 tons
Effective depth	4′-4″
,, width	4′-3″
Wheelbase of crab	5′-0″

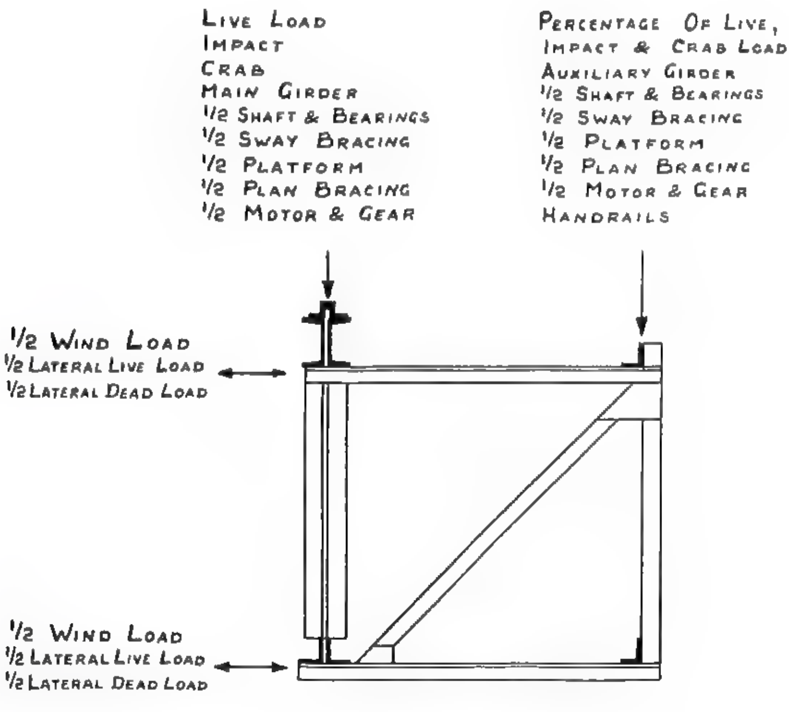


Fig. 4.

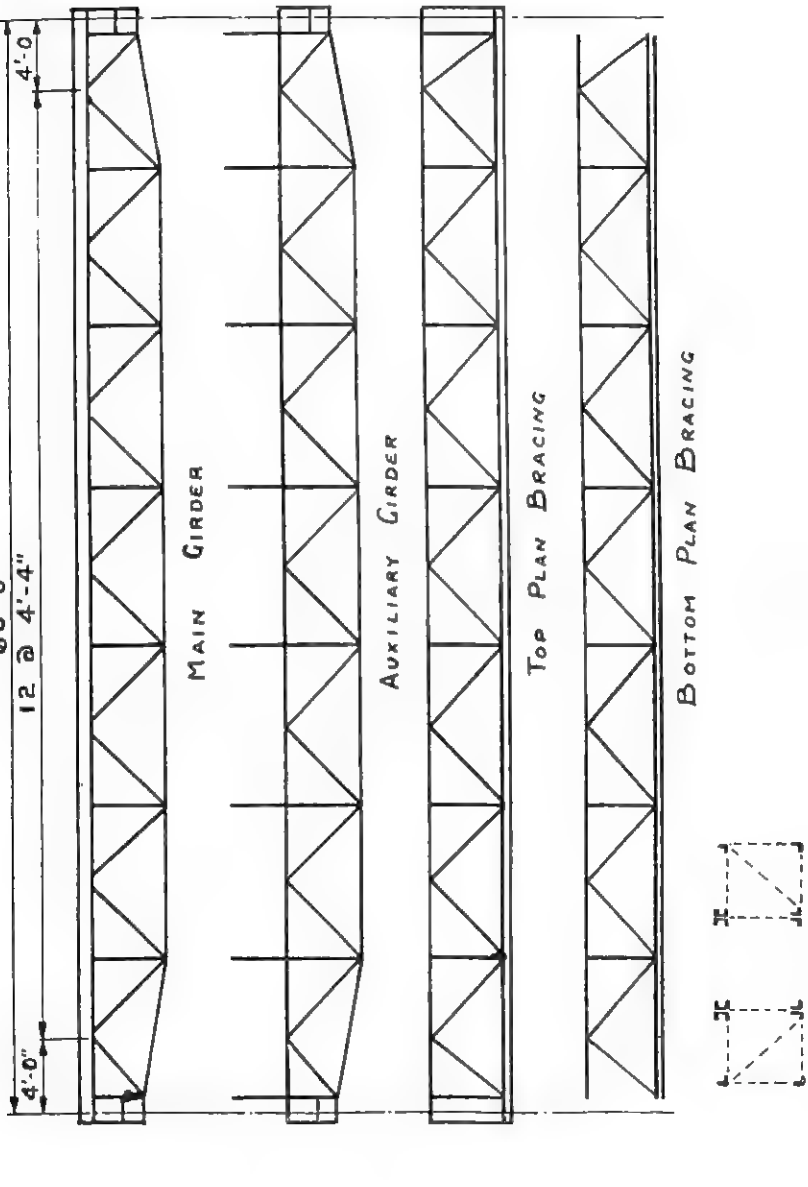

60'-0"
12 @ 4'-4"
4'-0
4'-0"
MAIN GIRDER
AUXILIARY GIRDER
TOP PLAN BRACING
BOTTOM PLAN BRACING

Fig. 5.

Distributed Dead Loads.

The section of the girders is shown in Fig. 4, from which it can be seen that the distributed dead loads are carried by the main and auxiliary as follows :—

	Main Girder	*Auxiliary Girder*
Main girder	3·50 tons	—
Auxiliary girder	—	0·9 tons
Plan bracings	0·25 ,,	0·25 ,,
Shafts and bearings ...	0·5 ,,	0·5 ,,
Platform	0·3 ,,	0·3 ,,
Handrails	—	0·05 ,,
	4·55 tons	2·00 tons

Distributed bending moments.

$$\text{Main girder} \quad \frac{4{\cdot}55 \times 60}{8} = 34{\cdot}1 \text{ tons ft.}$$

$$\text{Auxiliary girder} \quad \frac{2 \times 60}{8} = 15{\cdot}0 \text{ tons ft.}$$

Point Load.

The long travel motor will be considered to act as a point load at the centre of the span equally spread between both girders.

$$\text{BM on each girder} = \frac{0{\cdot}7 \times 60}{2 \times 4} = 5{\cdot}2 \text{ tons ft.}$$

These dead load bending moments are shown in Fig. 6, the values of the bending moments at the intermediate panel points can be scaled from the diagrams.

Live Loading.

The bending moments from live loads will be calculated by taking moments, see Fig. 7. In this example the top boom will be of uniform section throughout its length, therefore it will only be necessary to calculate the boom load in member 4-5. The bottom boom will consist of angles and a plate which will be curtailed where possible, therefore it will be necessary to calculate the boom loads in all panels.

From the theory of structures it is known that the boom load

$$\text{in member } 4\text{-}5 = \frac{\text{B.M. at point 8}}{\text{Effective depth}}$$

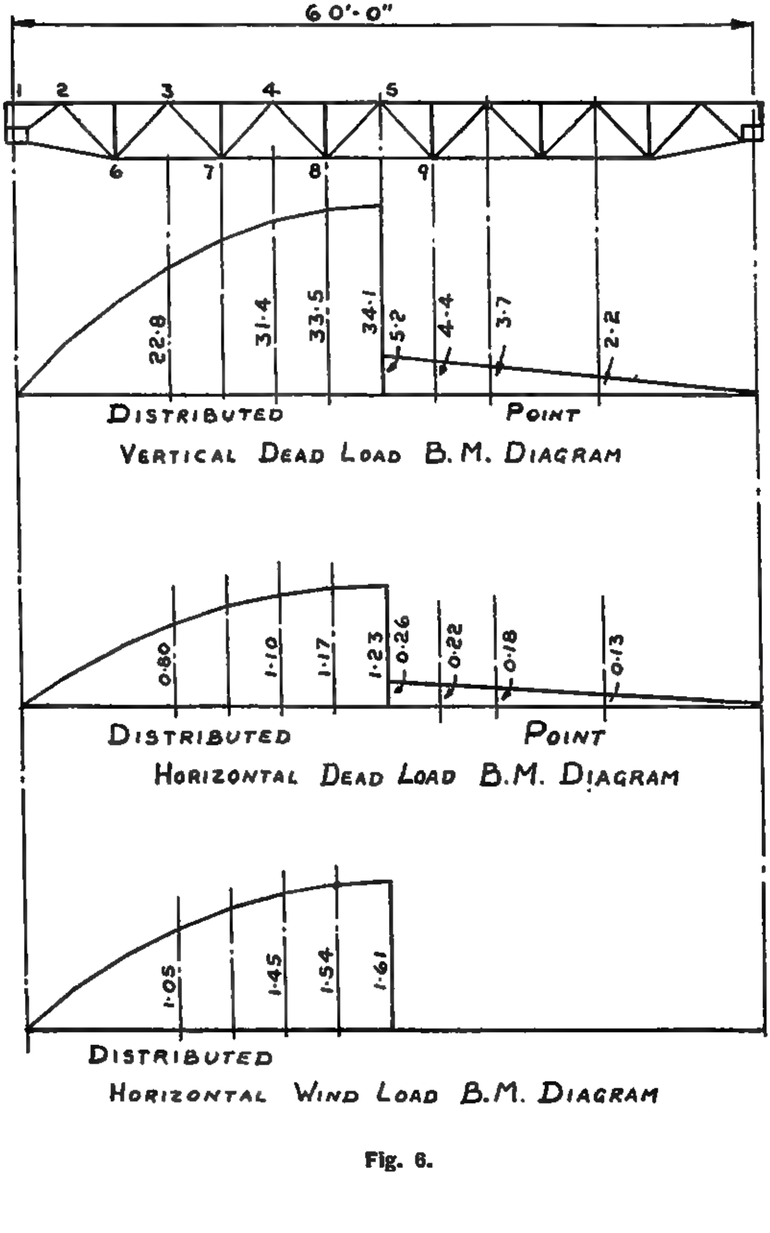

60'-0"
1
2
3
4
5
6
7
8
9
22·8
31·4
33·5
34·1
5·2
4·4
3·7
2·2
DISTRIBUTED
POINT
VERTICAL DEAD LOAD B.M. DIAGRAM
0·80
1·10
1·17
1·23
0·26
0·22
0·18
0·13
DISTRIBUTED
POINT
HORIZONTAL DEAD LOAD B.M. DIAGRAM
1·05
1·45
1·54
1·61
DISTRIBUTED
HORIZONTAL WIND LOAD B.M. DIAGRAM

Fig. 6.

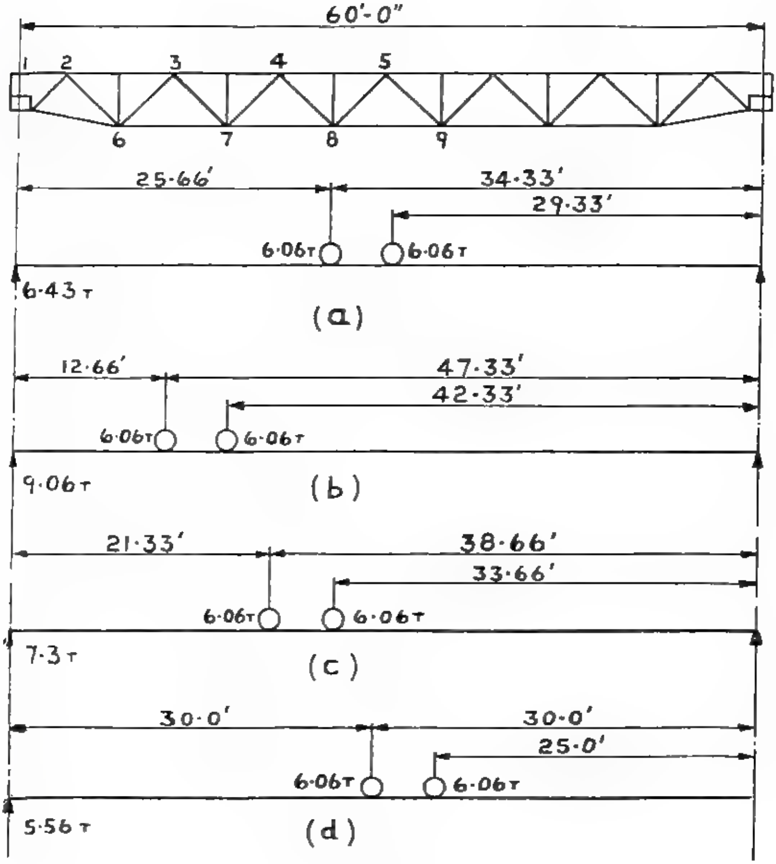

60'-0"
1
2
3
4
5
6
7
8
9
25·66'
34·33'
29·33'
6·06T
6·06T
6·43T
(a)
12·66'
47·33'
42·33'
6·06T
6·06T
9·06T
(b)
21·33'
38·66'
33·66'
6·06T
6·06T
7·3T
(c)
30·0'
30·0'
25·0'
6·06T
6·06T
5·56T
(d)

Fig. 7.

The imposed live loads consist

Load (including impact)	$= 15 \times 1{\cdot}35 =$	20·25 tons
Crab	$=$	4·0 ,,
		24·25 tons

the impact factor being 1·35 for Class 2 cranes. This load will be considered to be equally spread between the four crab wheels giving the wheel load of 6·06 tons. The wheels will be positioned as in Fig. 7 (*a*) for maximum bending moment at point 8.

$$\text{The left reaction} = \frac{6{\cdot}06\ (29{\cdot}33 + 34{\cdot}33)}{60} = 6{\cdot}43 \text{ tons.}$$

$$\therefore \quad \text{B.M. at Point 8} = 6{\cdot}43 \times 25{\cdot}66 = 165 \text{ tons ft.}$$

By similar calculations the bending moments at points 3, 4 and 5 will, when divided by the effective depth, give the boom loads in members 6-7, 7-8 and 8-9 respectively. See Fig. 7 (*b*), (*c*) and (*d*).

B.M. at Point 3 = 115 tons ft.
,, ,, ,, 4 = 156 ,, ,,
,, ,, ,, 5 = 167 ,, ,,

The proportion of the live load bending moments carried by the main and auxiliary girders, and the forces induced in the horizontal bracing by the live load, can be obtained by treating the structure as a space frame, and it is suggested that the papers listed in the bibliography are studied on this subject.

If the structure is a perfect space frame, the live load applied to the main girder deflects it and, in turn, pulls down the auxiliary girder a small amount, thus the live load is distributed in a certain proportion between each plane of the girder system as a whole.

However, a perfect frame is rarely obtained in practice, as it is necessary in many designs to omit a secondary bracing in order to accommodate either a mechanical or electrical component of the crane, thus upsetting the continuity of the bracings.

Each omission of bracing has to be treated separately in detailing by replacing with plated sections or bulk heads the true effectiveness of these being very indeterminate.

At the design stage of the girders the exact size of motors and electrical equipment is not always known, and therefore a compromise between perfect and imperfect frames must be made to enable the design to progress as soon as possible after receiving the order for the crane.

The following recommendations are made for the distribution of the live loads upon the girders.

(a) The main girder to be designed to carry 100% of the vertical live load bending moment. If the frame is imperfect the loading is correct, but if the frame is perfect the loading is slightly heavy.

(b) The auxiliary girder to be designed to carry a percentage of the live load, thus being correct for the perfect frame and slightly heavy for the imperfect frame.

(c) The bracing in the horizontal plane should be designed to carry the same live load percentage as the auxiliary girder. This percentage can only be determined after an approximate section has been assumed or by calculations carried out on past designs and from these it has been determined that the percentage is approximately 19% for 10 ton and 16% for 15 ton cranes.

At the completion of this design a comparison will be made using this example as a space frame.

Lateral Live Loads.

The lateral live loads applied at rail level cause the girder to rotate and bending moment is equally divided between the two horizontal lattice girders. The value of the lateral force is $\frac{1}{20}$ of safe working load and crab, assuming the long travel speed of 300 f.p.m.

Lateral live load bending moments per boom (by proportion of the vertical loading).

Member 4 - 5 $\dfrac{165 \times (15+4)}{24{\cdot}25 \times 2 \times 20} = 3{\cdot}23$ tons ft.

Member 6 - 7	2·25 ,, ,,
Member 7 - 8	3·06 ,, ,,
Member 8 - 9	3·27 ,, ,,

Lateral Dead Loads.

The lateral dead loads are also equally divided between the two horizontal lattice girders.

The total distributed dead loads from the data = 6·55 tons.

Lateral dead load (distributed) bending moments.

Member 8 - 9 $\dfrac{6{\cdot}55 \times 60}{20 \times 2 \times 8} = 1{\cdot}23$ tons ft.

Member 4 - 5	by proportion or scale.	1·17 ,, ,,
Member 7 - 8		1·10 ,, ,,
Member 6 - 7		0·80 ,, ,,

The total concentrated load from data = 0·7 tons.

Lateral dead concentrated load bending moment.

Member 8 - 9 $\quad \dfrac{0{\cdot}7 \times 60}{4 \times 2 \times 20} = 0{\cdot}26$ tons ft.

Member 4 - 5	by proportion or scale.	0·22	,, ,,
Member 7 - 8		0·18	,, ,,
Member 6 - 7		0·13	,, ,,

Wind Loads.

The wind loads are considered as 5 lbs. per sq. ft. acting on a loaded crane, or 30 lbs. per sq. ft. with the crane unloaded.

The area of the girder is taken as $1\frac{1}{2}$ times the projected area of members in the vertical plane and the load obtained distributed equally between the two horizontal girders.

The area of the crab is taken as the projected area and wind load imposed acts equally on each horizontal girder.

Projected area of girder = 130 sq. ft.

Distributed load $= \dfrac{5 \times 130 \times 1{\cdot}5}{2240} = 0{\cdot}43$ tons.

Area of crab = 28 sq. ft.

Live load $= \dfrac{28 \times 5}{2240} = 0{\cdot}06$ tons or 0·015 tons per wheel.

Lateral distributed wind load bending moments.

Member 8 - 9 $\quad \dfrac{0{\cdot}43 \times 60}{8 \times 2} = 1{\cdot}61$ tons ft.

Member 4 - 5	by proportion or scale	1·54	,, ,,
Member 7 - 8		1·45	,, ,,
Member 6 - 7		1·05	,, ,,

Lateral bending moments from wind on crab in proportion of vertical live loads.

Member 4 - 5	0·21 tons ft.
Member 8 - 9	0·21 ,, ,,
Member 7 - 8	0·19 ,, ,,
Member 6 - 7	0·14 ,, ,,

Top Boom of Main Girder.

Various types of booms are used in lattice girders and a selection of these is shown in Fig. 8.

Type (*a*). This is suitable only for very light cranes.

Type (*b*). Heavy stress concentrations occur in the extreme lower fibre of the plate and this section is not recommended.

Types (*c*), (*d*) *and* (*e*). These are very good sections for medium capacity cranes up to 30 tons safe working loads.

Type (*f*). This section is a good one for cranes of above 30 tons capacity.

Types (*g*) *and* (*h*). These are poor sections as they are expensive to manufacture and the stress distribution is poor.

Type (*i*). A very good section for cranes of 100 tons capacity and over.

The section of the top boom in this example will be of type (*e*) and will consist of $2 - 8'' \times 3''$ channels, $1 - 8'' \times \frac{3}{8}''$ top plate, and a 56 lbs. per yard bridge rail.

The rail will not be considered in the section.

The inertias, moduli, radii of gyration and area of section can be obtained by normal methods resulting as follows :—

I_{XX}	133·3 inch^4
Z_C	39·69 inch^3
Z_T	26·59 inch^3
Z_{YY}	8·22 inch^3
k_{YY}	1·34 inch
k_{XX}	3·16 inch
Area	12·38 sq. in.

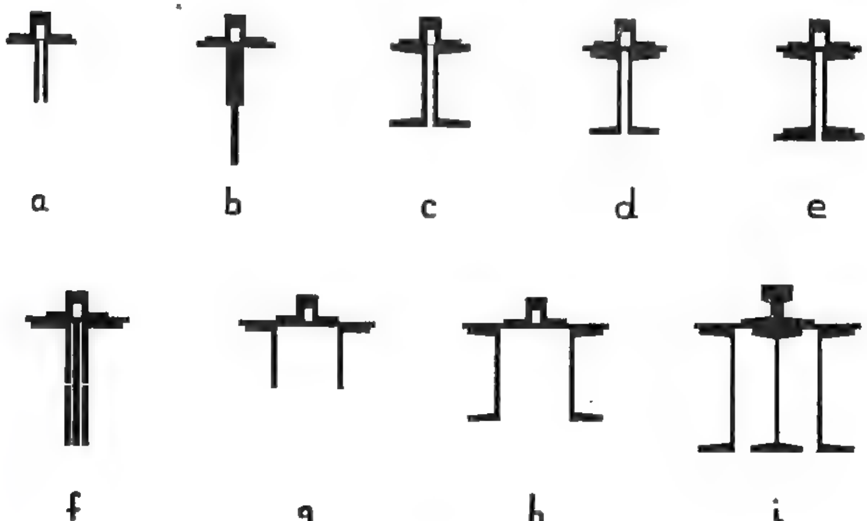


Fig. 8.

The remaining properties necessary to obtain allowable stresses are as follows :—

t_e = effective thickness of flange or mean thickness.

$$\therefore \quad t_e = \frac{\text{area of horizontal portion of flange}}{\text{width}}$$

$$= \frac{(8'' \times \tfrac{3}{8}'') + (2 \times 3 \times 0{\cdot}44)}{8} = 0{\cdot}71''$$

$$M = \frac{\text{Moment of inertia of compression flange about Y-Y axis of the girder}}{\text{Moment of inertia of whole section about YY axis of the girder}}$$

For the above section M = 0·74.

K_2 is a value which allows for inequality of tension and compression flange inertias and varies according to values of M.

For values of M of 0·5 or greater $K_2 = 0$. Therefore for this section $K_2 = 0$.

Boom Loads.

Summary of vertical bending moments for member 4 - 5.

Live load	165·0	tons ft.
Distributed dead load ...	33·5	,, ,,
Point dead load	4·4	,, ,,
Total ...	202·9	,, ,,

Effective depth = 4·33 ft.

Boom load from vertical B.M. $= \frac{202{\cdot}9}{4{\cdot}33} = 46{\cdot}8$ tons.

Summary of lateral bending moments for member 4 - 5.

Live load	3·23	tons ft.
Distributed dead load ...	1·17	,, ,,
Point dead load	0·22	,, ,,
Distributed wind load ...	1·54	,, ,,
Crab wind load	0·21	,, ,,
Total ...	6·37	,, ,,

Effective width = 4·25 ft.

Boom load from lateral B.M. $= \frac{6{\cdot}37}{4{\cdot}25} = 1{\cdot}5$ tons.

Total boom load $= 46{\cdot}8 + 1{\cdot}5 = 48{\cdot}3$ tons.

Compressive stress $(f_a) = \dfrac{48{\cdot}3}{12{\cdot}38} = 3{\cdot}9$ tons per sq. in.

Length of unsupported strut $(l) = 104$ ins.

Slenderness ratio $= \dfrac{l}{k_{yy}} = 78.$

The strut is continuous over supports therefore the effective length $= 0{\cdot}85\,l.$

Effective $\dfrac{l}{k_{yy}} = 66.$

The allowable compressive stress $(F_a) = 5{\cdot}4$ tons per sq. in. obtained by interpolation of Table 1 in the Appendix.

$$\frac{f_a}{F_a} = 0{\cdot}722$$

The crab wheel positioned between panel points applies both a local vertical and horizontal bending moment to the top boom section and this is considered equal to $\dfrac{WL}{6}$ between the panel points and $\dfrac{WL}{12}$ at the panel point.

Where W = wheel load.
L = span between supports.

Local bending moments.

Centre of panel $= \dfrac{6{\cdot}06 \times 52}{6} = 52{\cdot}6$ tons in.

Panel point $= \dfrac{6{\cdot}06 \times 52}{12} = 26{\cdot}3$ tons in.

With the wheel positioned for maximum vertical local bending, there is also a lateral local bending stress over a span of 104 inches between horizontal plan bracing.

Thus the first wheel is positioned 26 inches from the left-hand support and the second wheel 86 inches from the left-hand support.

Lateral wheel load $= \dfrac{19}{4 \times 20} = 0{\cdot}238$ tons.

Left-hand reaction $= \dfrac{0{\cdot}238\ (78 + 18)}{104} = 0{\cdot}22$ tons.

Lateral local B.M. $0{\cdot}22 \times 26 \times \frac{2}{3}$ (for continuity) $= 3{\cdot}8$ tons in.

Bending Stress at Centre of Panel.

Compressive vertical bending stress $= \frac{52 \cdot 6}{39 \cdot 69} = 1 \cdot 33$ tons per sq. in.

Compressive lateral bending stress $= \frac{3 \cdot 8}{8 \cdot 22} = 0 \cdot 46$ tons per sq. in.

Total bending stress (f_{bc}) = 1·79 tons per sq. in.

The allowable basic bending stress $F_{bc} = A + K_2 B$ tons per sq. in. or 10 tons per sq. in., whichever is the least.

Where the values of A and B are obtained from Table 2 in terms of l/k_y and d/t_e, l/k_y has been previously determined as 66, d/t_e = 8·375/0·71 = 12.

Where d = depth of boom.

t_e = effective thickness of flange.

Thus the basic bending stress = 10 tons per sq. in.

To this has to be applied the stress factor which for Class 2 cranes = 0·8.

∴ Allowable bending stress = 0·8 × 10 = 8 tons per sq. in.

$$\frac{f_{bc}}{F_{bc}} = \frac{1 \cdot 79}{8} = 0 \cdot 22.$$

Members subject to combined bending and compression should comply with the following :—

$$\frac{f_a}{F_a} + \frac{f_{bc}}{F_{bc}} \text{ does not exceed unity}$$

$$0 \cdot 722 + 0 \cdot 22 = 0 \cdot 942.$$

Stresses at Panel Point.

Compressive vertical bending stress $f_{bc} = \frac{26 \cdot 3}{26 \cdot 59}$

$= 0 \cdot 99$ tons per sq. in.

As this is less than the centre of panel stress, we need not proceed further.

Bottom Boom Main Girder.

It is best to consider the whole length of this boom simultaneously in order to select a section best suited for the complete length.

Vertical Loading.

	Members. 8-9	7-8	6-7
B.M. live load	167	156	115
,, Distributed dead load	34·1	31·4	22·8
,, Point dead load	5·2	3·7	2·2
Total B.M. ...	206·3	191·1	140·0
Boom load	47·5 tons	44 tons	32·3 tons

Lateral Loading.

B.M. live loads	3·27	3·06	2·25
„ distributed dead load	1·23	1·10	0·80
„ point dead load	0·26	0·18	0·13
„ distributed wind load	1·61	1·45	1·05
„ crab wind load	0·21	0·19	0·14
	6·58	5·98	4·37
Boom load	1·55 tons	1·41 tons	1·03 tons
Total boom load ...	49·05 tons	45·41 tons	33·33 tons

The allowable tensile stress is given in B.S. 2573 as a basic stress of 9 tons per sq. in. This, of course, has to be multiplied by the stress factor for Class 2 cranes of 0.8. Thus the allowable tensile stress is 7·2 tons per sq. in.

The procedure is to select several angles and plates, calculate the safe load on each section, and then by trial and inspection select the economical section.

$2 - 3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ Angles.

Gross area = 4·98 sq. in.	
Less $2 - \frac{13}{16}''$ rivet holes	0·61 sq, ins.
Nett area	4·27 sq. ins.
Safe load	30·7 tons.

$2 - 4'' \times 4'' \times \frac{3}{8}''$ Angles.

Gross area	5·72 sq. ins.
Less $2 - \frac{13}{16}''$ rivet holes	0·61 sq. ins.
Nett area	5·11 sq. ins.
Safe load	36·8 tons.

$1 - 9'' \times \frac{3}{8}''$ Flat.

Gross area	3·375 sq. ins.
Less $2 - \frac{13}{16}''$ rivet holes	0·61 sq. ins.
Nett area	2·765 sq. ins.
Safe load	19·9 tons.

From the above safe loads, it will be seen that if $2 - 4'' \times 4'' \times \frac{3}{8}''$ angles are selected, a $9'' \times \frac{3}{8}''$ flat will be required over members 7 - 8 and 8 - 9. Whereas if $2 - 3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles are used, the $9'' \times \frac{3}{8}''$ flat will need to extend over members 6 - 7, 7 - 8 and 8 - 9.

By comparing the weights of the two sections it will be seen that the $4'' \times 4'' \times \frac{3}{8}''$ angles are the most economical.

Top Boom of Auxiliary Girder.

This usually consists of an angle section, with one leg at least 4″ deep, which in most cases will eliminate gusset plates in that plane.

Vertical bending moments.

Live load 16% × 165 = 26·4 tons ft.

Distributed dead load $15 \times \frac{33\cdot5}{34\cdot1}$ = 14·7 „ „

Point dead load 4·4 „ „

45·5 „ „

The effective depth of the auxiliary girder = 4′-2¼″.

Vertical boom load = $\frac{45\cdot5}{4\cdot26}$ = 10·7 tons.

The lateral boom load will be as the main girder, *i.e.*, 1·5 tons.

Total boom load = 12·2 tons.

Section 4″ × 3″ × ⅜″ angle.

Area 2·49 sq. ins.

Minimum radius of gyration = 0·64″.

Compressive stress = $\frac{12\cdot2}{2\cdot49}$ = 4·9 tons per sq. in.

Unsupported length of strut = 52 ins.

Slenderness ratio = $\frac{52}{0\cdot64}$ = 81

The strut is continuous over supports therefore effective length = 0·85 L.

Effective slenderness ratio = 69

Allowable compressive stress = 5·28 tons per sq. in.

Bottom Boom of Auxiliary Girder.

Vertical bending moments.

Live load 16% × 167 = 26·7 tons ft.

Distributed dead load 15 „ „

Point dead load 5·2 „ „

46·9 „ „

Boom load due to vertical B.M. = 11·0 tons.

„ „ „ „ lateral B.M. = 1·55 „

Total boom load 12·55 „

Section $4'' \times 3'' \times \frac{5}{16}''$ angle.

Gross area	2·09 sq. in.
Less $1-\frac{13}{16}''$ hole	0·22 ,, ,,
Nett area	1·87 ,, ,,

$$\text{Tensile stress} = \frac{12{\cdot}55}{1{\cdot}87} = 6{\cdot}7 \text{ tons per sq. in.}$$

Allowable tensile stress = 7·2 tons per sq. in.

Shop Joints in Booms.

The joint in the individual members of the top boom should be staggered so that they do not occur on the same line.

The joint in the $8'' \times \frac{3}{8}''$ flat is made by welding and the rail is included as a cover plate in the channel joint.

It is recommended that no less than three rivets in line on any leg or flange of angle or channel should be used.

Site Joint in Booms.

Where access to site is difficult it is often necessary to make a bolted joint in the complete girder.

All bolted joints should be designed to carry the boom load plus 25%.

The bolts should be barrel fit bolts, as shown in Fig. 9, the holes for which will be the same size as rivet holes. This method protects the bolt thread in making and breaking the joint and also provides a hole suitable for rivets if site rivetting is required.

Fig. 10 shows both the shop and site joints of all the boom sections.

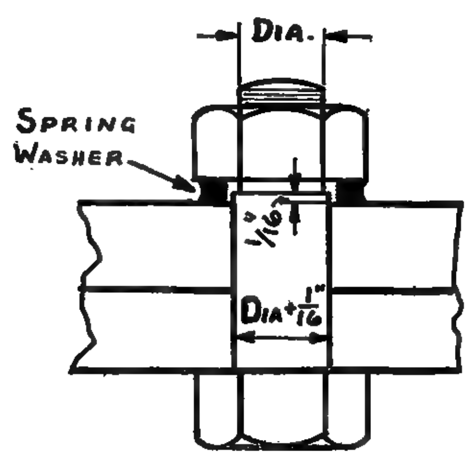


Fig. 9.

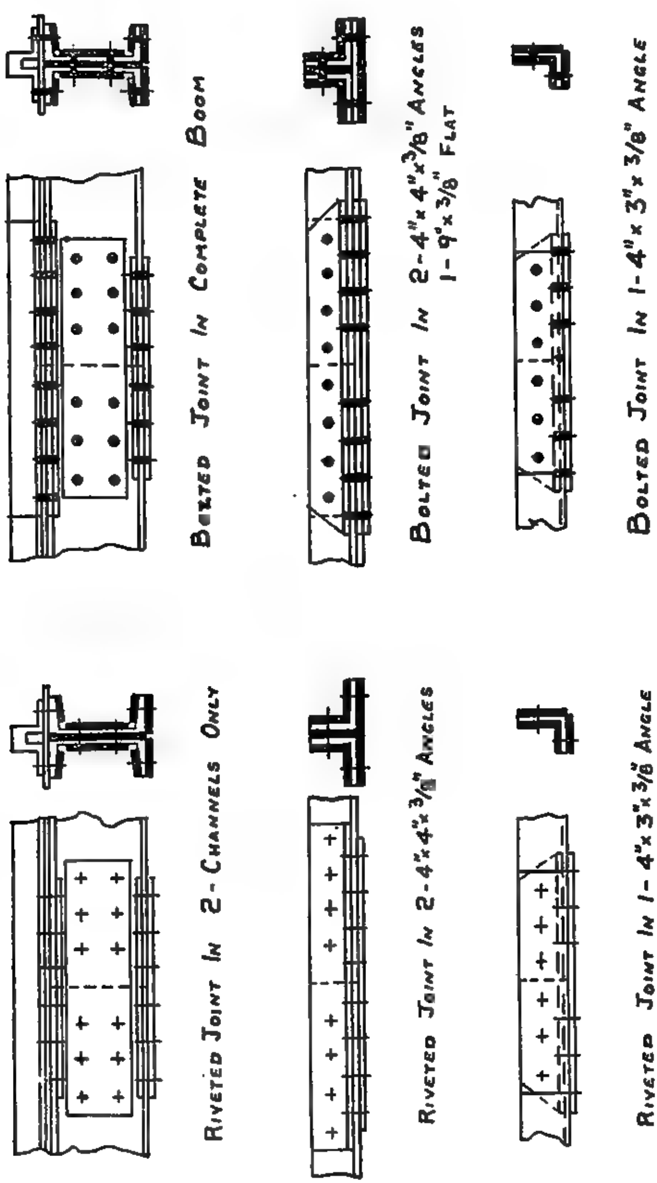

Bolted Joint In Complete Boom
Bolted Joint In 2-4" x 4" x 3/8" Angles
1-9" x 3/8" Flat
Bolted Joint In 1-4" x 3" x 3/8" Angle
Riveted Joint In 2-Channels Only
Riveted Joint In 2-4" x 4" x 3/8" Angles
Riveted Joint In 1-4" x 3" x 3/8 Angle

Fig. 10.

Main Girder Web Members.

A method of obtaining the loads in the web members is shown in Fig. 11. The maximum shear diagram is drawn for the live load above the base line A-B by the following method, assuming each wheel loaded equal.

(*a*) Measure to scale two times the value of the wheel load, at a distance of half the wheel base to the left of the left-hand reaction (line A - C).

(*b*) Join point C to a point D fixed at half the wheel base from B.

(*c*) Insert line EB where E is positioned a distance from B equal to the wheelbase of the crab.

The shear for live load for any point across the span is measured vertically at that point between the base line A - B and the shear diagram line C.E.B. Proof of the above can be found in most structural textbooks.

The dead load shear diagram is drawn in the normal manner as shown, but with the shear to the left of the centre line below the base line and the shear to the right of the centre above the base line. This enables shear values to be obtained by direct scale.

The dead load shear consists of distributed loading and central point load.

Distributed loads 4·55 tons, *i.e.*, 2·28 tons reaction.

Point load. 0·7/2 = 0·35 tons, *i.e.*, 0·18 tons reaction.

Load Reversal.

The shear diagram for live loads gives the value of the shear at the point considered and also the reaction value ; this diagram is drawn for the load moving from B to A.

A similar but reverse diagram could be drawn for the load moving from A to B, but to save drawing this other diagram, the right hand half of the girder can be used if the live load shear to the right of the centre line is considered to have altered its sign from positive to negative. This is of opposite sign to the dead load, thus the value required is the difference between dead and live shears.

As an example consider member 4 - 8, with the load moving from right to left the maximum shear occurs when the leading wheel reaches point 4*a*, this equals :—

Live load ...	... ...	6·42 tons
Point load	... ...	0·18 ,,
Distributed load	...	0·35 ,,
		6·95 ,,

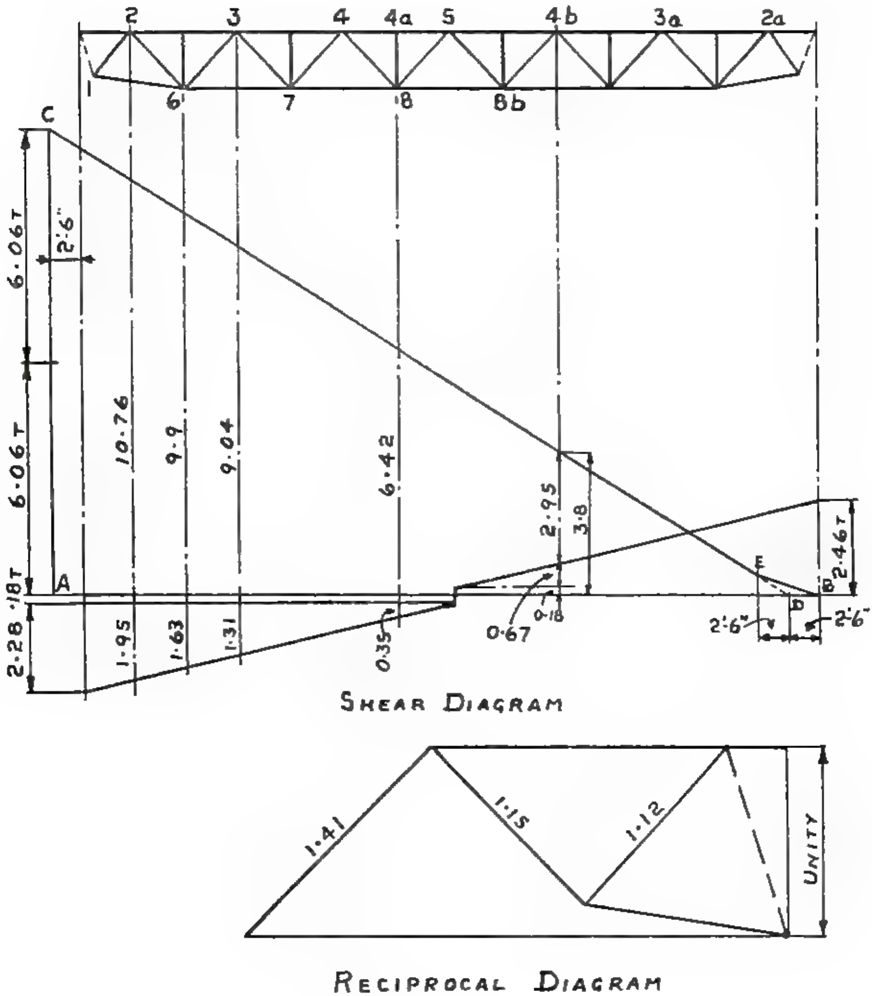

2
3
4
4a
5
4b
3a
2a
1
6
7
8
8b
C
2·6"
6·06T
6·06T
10·76
9·9
9·04
6·42
2·95
3·8
2·46T
·18T
A
E
B
D
2·28
1·95
1·63
1·31
0·35
0·18
0·67
2·6"
2·6"
SHEAR DIAGRAM
1·41
1·15
1·12
UNITY
RECIPROCAL DIAGRAM

Fig. 11.

With the load moving from left to right the maximum shear occurs when the leading wheel reaches point 4, this is equivalent to point 4*b* in the other half of the diagram and from this must be subtracted the dead load.

Live load ...	... ...	$-3{\cdot}8$ tons
Point load	... ...	$+0{\cdot}18$,,
Distributed load	...	$+0{\cdot}67$,,
		$2{\cdot}95$,,

Reciprocal Diagram.

A reciprocal diagram for unit shear is drawn and the values of all members scaled. These values or shear co-efficients when multiplied by the vertical shear loads obtained produce the value of the load in the member.

Load Sign in Members.

A simple rule to determine if a member is subject to compressive or tensile loads is that if a load is imagined to be suspended from the centre of the bottom boom, then in all members that point downwards to this load the major load is tensile and in all members that point upwards the major load is compressive.

Thus in members 1 - 2, 6 - 3, 7 - 4 and 8 - 5, the major load is compressive whilst the major load is tensile in members 2 - 6, 3 - 7 and 4 - 8.

Member 1 - 2.

Shear	$12{\cdot}89$ tons
Coefficient	$1{\cdot}12$
Compressive load $12{\cdot}89 \times 1{\cdot}12$	$=$ $14{\cdot}5$ tons
Length	55 ins.
Effective length $0{\cdot}8 \times 55$	$=$ 44 ins.
Section	$2 - 3'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$ angles
Effective l/k_{yy}	$=$ 48
Area	$=$ $3{\cdot}24$ sq. in.

Allowable stress (see Table 1, Class 2) $6{\cdot}03$ tons per sq, in.

Safe load $6{\cdot}03 \times 3{\cdot}24 = 19{\cdot}5$ tons.

The rivets are in double shear and bearing, using a $\frac{3}{8}''$ gusset plate with $\frac{3}{4}''$ diameter rivets, the bearing value of $3{\cdot}38$ tons per rivet (see Table 3, Class 2) is the deciding figure.

Number of rivets required $= \dfrac{12{\cdot}89}{3{\cdot}38}$. Say 4.

Member 2 - 6.

Shear	11·7 tons
Coefficient	1·15
Tensile load $= 11{\cdot}7 \times 1{\cdot}15$	$=$ 13·4 tons
Section	$2 - 3'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$ angles
Gross area	3·24 sq. in.
$2 - \frac{13}{16}''$ rivet holes	0·51 ,, ,,
Nett area	2·73 ,, ,,
Allowable stress $= 9 \times 0{\cdot}8$	$=$ 7·2 tons per sq. in.
Safe load $7{\cdot}2 \times 2{\cdot}73$	$=$ 19·6 tons
No. of rivets at 3·38 tons bearing each	$= \dfrac{13{\cdot}4}{3{\cdot}38}$ Say $4 - \frac{3}{4}''$ dia.

Member 3 - 6.

When the bottom boom slopes upwards at the end of the girder the member 3 - 6 is usually the heaviest load and longest strut, thus it is the deciding member.

Shear	10·53 tons
Coefficient	1·41
Compressive load. $10{\cdot}53 \times 1{\cdot}41$	$=$ 14·9 tons
Length	73·2″
Effective length. $73{\cdot}2 \times 0{\cdot}8$	$=$ 58·6″
Section	$2 - 3'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$ angles
Effective slenderness ratio	$=$ 63
Area	3·24 sq. in.

Allowable stress (see Table 1, Class 2) $=$ 5·5 tons per sq. in.

Safe load $= 5{\cdot}5 \times 3{\cdot}24 = 17{\cdot}8$ tons

No. of rivets at 3·38 tons bearing each $= \dfrac{14{\cdot}9}{3{\cdot}38}$. Say $5 - \frac{3}{4}''$ dia.

In a girder of this capacity and span it is advisable to keep all the members in the same section to help in production.

However, with spans of over 80′-0″ or of heavier tonnages, two or three various sections prove both economical and practical. This change of sections can only be left to the discretion and experience of the designer.

Where all the members are of the same section, it is advisable to keep the number of rivets in each member equal, thus the templates required for the gusset plates are kept to a minimum.

If any attempt is made to reduce the sections towards the centre of the span, care should be taken to allow for stress reversals.

As an example of the calculations required for a member subject to reversals of stress, member 4 - 8 will be checked.

Member 4 - 8.

A member subject to reversal of stress should be proportioned to withstand the whole major load plus one half of the minor load.

Tensile shear		6·95 tons
Compressive shear		2·95 tons
Coefficient		1·41
Tensile load.	$6{\cdot}95 \times 1{\cdot}41$	= 9·8 tons
Compressive load.	$2{\cdot}95 \times 1{\cdot}41$	= 4·16 tons
Section		$2 - 3'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$ angles.
Allowable tensile stress		7·2 tons per sq. in.
Allowable compressive stress		5·51 tons per sq. in.

$$\text{Area required} = \frac{6{\cdot}95}{7{\cdot}2} + \frac{2{\cdot}95}{2 \times 5{\cdot}51} = 1{\cdot}23 \text{ sq. in.}$$

Area of section 3·24 sq. in.

No. of rivets required ($\frac{3}{4}''$ dia. at 3·38 tons bearing)

$$= \left(6{\cdot}95 + \frac{2{\cdot}95}{2}\right) \div 3{\cdot}38. \quad \text{Say } 3 - \tfrac{3}{4}'' \text{ dia. rivets.}$$

Vertical Members—Main Girder.

If the panel length is less than the wheel base of the crab, the load carried by the vertical member is equal to the crab wheel load.

If the panel length is greater than the wheel base of the crab, the load carried by the vertical member is the greater wheel load plus a proportion of the lesser wheel load, obtained by taking moments.

In the example the panel length is less than the wheel base therefore :—

Load	6·06 tons
Section	$2 - 2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$ angles
Length	52 inches

Effective length $= 52 \times 0{\cdot}8 = 42$ inches

$$\text{Effective slenderness ratio} = \frac{42}{0{\cdot}75} = 56$$

Area 2·93 sq. in.

Allowable stress (see Table 1, Class 2) = 5·77 tons per sq. in.

Safe load. $2{\cdot}93 \times 5{\cdot}77 = 16{\cdot}9$ tons

No. of rivets ($\frac{3}{4}''$ dia., 3·38 tons bearing) $\frac{6{\cdot}06}{3{\cdot}38}$. Say 2.

Auxiliary Girder Web Members.

Usually the shear members of the auxiliary girder have little load to carry in an ordinary standard crane and it is only sufficient to decide the section of the member by keeping it within a maximum slenderness ratio of 180.

Checking the strength of member 3 - 6 as the long heaviest loaded strut. The loads can be obtained by proportion from Fig. 11 by the rates of the load carried by main and auxiliary girders.

Live load shear..	$16\% \times 9{\cdot}04$	$= 1{\cdot}45$ tons
Distributed dead load shear	$\frac{2}{4{\cdot}55} \times 1{\cdot}31$	$= 0{\cdot}58$ tons
Point load shear		$= 0{\cdot}18$ tons
		$2{\cdot}21$ tons

Coefficient		$1{\cdot}41$
Compressive load.	$1{\cdot}41 \times 2{\cdot}21$	$= 3{\cdot}12$ tons
Length		$= 50''$
Effective length.	$0{\cdot}8 \times 50$	$= 40''$
Section		$1 - 2\frac{1}{2}'' \times 2\frac{1}{2}'' = \frac{5}{16}''$ angle.
Effective slenderness ratio	$\frac{40}{0{\cdot}48}$	$= 84$
Area		$1{\cdot}46$ sq. in.
Allowable stress (see Table 1, Class 2)		$= 4{\cdot}63$ tons per sq. in.
Safe load		$6{\cdot}7$ tons

The above load is applied ecentrically, but as the stress is so small the eccentric stress need not be calculated.

The rivets will be $\frac{5}{8}''$ dia., bearing in $\frac{5}{16}''$ thick metal or single shear. The single shear value of 1·6 tons being the least value, decides the number of rivets as 2.

Horizontal Plan Bracing.

With cranes of capacities of under 50 tons the load in the plan bracing is generally not sufficient to decide a section, the criterion being that the slenderness ratio should not be less than 180.

In this example $2\frac{1}{4}'' \times 2\frac{1}{4}'' \times \frac{1}{4}''$ angles will be used with $2 - \frac{5}{8}''$ dia. rivets at each end ; for cranes of 25 ton to 50 ton capacity a $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$ angle should be used.

Internal Sway Bracing.

These members transfer part of the live load from the main to the auxiliary girder.

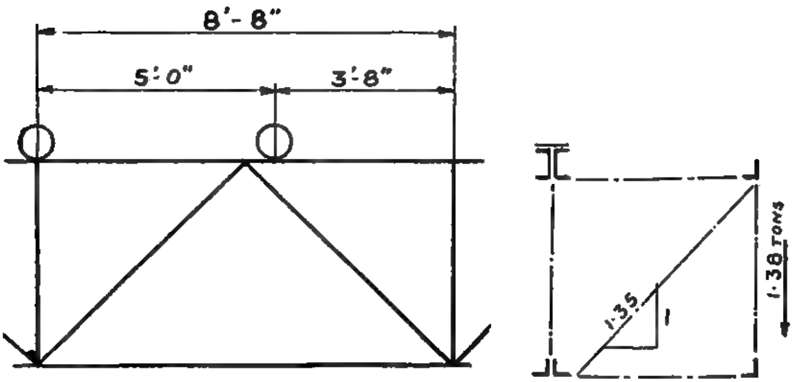


Fig. 12.

Vertical shear in bracing = 16% of the wheel load reaction at the panel point (see Fig. 12).

$$= 16\% \left(6{\cdot}06 + \frac{6{\cdot}06 \times 3{\cdot}66}{8{\cdot}66}\right)$$

$$= 1{\cdot}38 \text{ tons.}$$

The load is tensile if the member is attached at the top to the auxiliary girder.

Slope of member = 1·35/1 by scaling.

Load in member = $1{\cdot}35 \times 1{\cdot}38 = 1{\cdot}86$ tons.

Section $2\frac{1}{4}'' \times 2\frac{1}{4}'' \times \frac{1}{4}''$ angle.

Gross area = 1·06 sq. in.

$1 - \frac{11}{16}''$ hole $+ \frac{1}{2}$ outstanding leg = 0·42 ,, ,,

Nett area = 0·64 ,, ,,

Tensile stress $= \dfrac{1{\cdot}86}{0{\cdot}64} = 2{\cdot}9$ tons per sq. in.

Fig. 13 shows typical details of the main girder end.

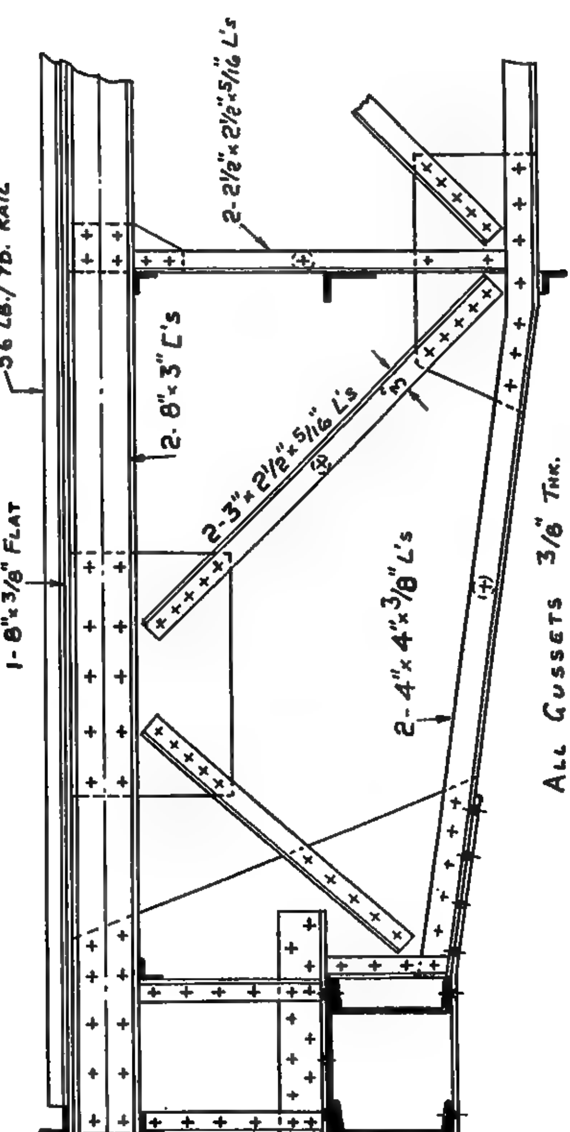

56 LB./YD. RAIL
2-2½"×2½"×5/16" L's
2-8"×3" L's
2-3"×2½"×5/16" L's
1-8"×3/8" FLAT
2-4"×4"×3/8" L's
ALL GUSSETS 3/8" THK.

Fig. 13.

The Bristol Iron and Steel Research Association has issued a report entitled *The Deflection Method of Design for E.O.T. Cranes,* which gives an approximate method of determining the loads in a lattice girder, braced on all four sides. The following formulae are used for the boom loads

$$P_A = -\frac{K_1}{d} M_W - \frac{K_3}{b} M_F$$

$$P_B = +\frac{K_1}{d} M_W - \frac{K_4}{b} M_F$$

$$P_C = -\frac{K_2}{d} M_W + \frac{K_3}{b} M_F$$

$$P_D = +\frac{K_2}{d} M_W + \frac{K_4}{b} M_F$$

Where	P	=	Boom load.
K_1, K_2, K_3 &	K_4	=	Factors obtained as below.
	M_W	=	Vertical bending moment of all loads.
	M_F	=	Lateral bending moment of all loads.
	d	=	depth of girder.
	b	=	width of girder.

$K_1 = A_A\ A_B\ (A_C + A_D)/C$
$K_2 = A_C\ A_D\ (A_A + A_B)/C$
$K_3 = A_A\ A_C\ (A_B + A_D)/C$
$K_4 = A_B\ A_D\ (A_A + A_C)/C$
$C = A_A A_B A_C + A_A A_B A_D + A_A A_C A_D + A_B A_C A_D$
A = Area of boom.

The suffixes A, B. C and D of the area A and the boom load P refer to the top main, bottom main, top auxiliary and bottom auxiliary boom respectively.

Using the previous design as an example we obtain the following :

A_A	12·38 sq. in.
A_B	9·095 ,, ,,
A_C	2·49 ,, ,,
A_D	2·09 ,, ,,

The area of the bottom flange is not constant over its full length and therefore should be modified as follows :—

$$A_B = \frac{(l_1^2 + l_1\ l_2)\ A_1\ A_2}{A_2\ l_1^2 + A_1\ l_1\ l_2}$$

Where A_1 = Area of boom not plated.
A_2 = Area of boom plated.

l_2 = Length of plated member.

$$l_1 = \frac{\text{Span} - l_2}{2}$$

From this we obtain A_8 (corrected) = 7·4 sq. in.

$C = 522$
$K_1 = 0·805$
$K_2 = 0·195$
$K_3 = 0·56$
$K_4 = 0·44$

M_w = total vertical bending moment, *i.e.*,

Distributed lead load main girder	34·1
,, ,, ,, auxiliary girder	15·0
Point lead load main girder	5·2
,, ,, ,, auxiliary girder	5·2
Live load	165·0
∴ M_w	224·5 tons ft.

F_w = total lateral bending moment.

Live load	2 × 3·23	=	6·46	tons ft.
Distributed dead load	2 × 1·17	=	2·34	,, ,,
Point dead load	2 × 0·22	=	0·44	,, ,,
Distributed wind load	2 × 1·54	=	3·08	,, ,,
Live wind load	2 × 0·21	=	0·42	,, ,,
∴ F_w		=	12·74	,, ,,

This load could act in either direction.

d = 4·33 ft.
b = 4·25 ft.

$$P_A = -\frac{0·825}{4·33} \times 224·5 \pm \frac{0·56}{4·25} \times 12·74$$

From which the maximum compression = 43·3 tons compared with 48·3 tons calculated in the previous example, and

$$\frac{f_a}{F_a} + \frac{f_{bc}}{F_{bc}} = 0·868$$

$$P_B = +\frac{0·805}{4·33} \times 224·5 \pm \frac{0·44}{4·25} \times 12·74$$

From which the maximum tension = 42·92 tons compared with 49·05 tons in previous example.

$$P_C = \frac{0·195}{4·33} \times 224·5 \pm \frac{0·56}{4·25} \times 12·74$$

$\therefore$ Maximum compression 11·78 tons compared with 12·2 tons previously calculated.

$$P_D = \frac{0{\cdot}195}{4{\cdot}33} \times 224{\cdot}5 \pm \frac{0{\cdot}44}{4{\cdot}24} \times 12{\cdot}74$$

$\therefore$ Maximum tension = 11·42 tons compared with 12·55 tons.

From this analysis it can be seen that the loads in the auxiliary girder booms in the previous example are very accurate, whereas the main girder was slightly overdesigned.

Checking the loading in the bracings the following formulae are used for the live loads at rail level :—

$$X_1 = \tfrac{1}{2}\left[W\left(1+K_1\right) \pm \frac{d}{b} K_4 F\right]$$

$$X_2 = \tfrac{1}{2}\left[W K_2 \pm \frac{d}{b} K_4 F\right].$$

$$X_3 = \tfrac{1}{2}\left[\pm F\left(1+K_3\right) - \frac{b}{d} K_2 W\right]$$

$$X_4 = \tfrac{1}{2}\left[\pm F K_4 + \frac{b}{d} K_2 W\right]$$

where X_1, X_2, X_3 and X_4 are the shears in the main girder, auxiliary girder, top horizontal bracing and bottom horizontal bracing respectively.

K_1, K_2, K_3, K_4, b and d are as calculated for the boom analysis.

W = Vertical shear from live load, *i.e.*, 10·76 tons in end panel.

F = Lateral shear from live load, *i.e.*, $\dfrac{10{\cdot}76}{20}$ = 0·538 tons in end panel.

$\therefore$ Shears in end panels of bracing due to live loading is

$$X_1 = \tfrac{1}{2}\left[10{\cdot}76\left(1+0{\cdot}805\right) \pm \frac{4{\cdot}33}{4{\cdot}25}\times 0{\cdot}44 \times 0{\cdot}538\right] = 9{\cdot}84 \text{ tons}$$

$$X_2 = \tfrac{1}{2}\left[10{\cdot}76 \times 0{\cdot}195 \pm \frac{4{\cdot}33}{4{\cdot}25}\times 0{\cdot}44 \times 0{\cdot}538\right] = 1{\cdot}17 \text{ tons}$$

$$X_3 = \tfrac{1}{2}\left[\pm 0{\cdot}538\left(1+0{\cdot}56\right) + \frac{4{\cdot}25}{4{\cdot}33}\times 0{\cdot}195 \times 10{\cdot}76\right] = 1{\cdot}45 \text{ tons}$$

$$X_4 = \tfrac{1}{2}\left[\pm 0{\cdot}538 \times 0{\cdot}44 + \frac{4{\cdot}25}{4{\cdot}33}\times 0{\cdot}195 \times 10{\cdot}76\right] = 1{\cdot}15 \text{ tons.}$$

Adding the dead load shears obtained in the previous example to the live load shears of the main and auxiliary girders, the following results are obtained :—

Main girder :—$9{\cdot}84+0{\cdot}18+1{\cdot}95 = 11{\cdot}97$ tons compared with 12·89 tons used in previous example.

Auxiliary girder :—$1{\cdot}17+0{\cdot}58+0{\cdot}18=1{\cdot}93$ tons compared with 2·21 tons used in previous example.

The top plan bracing is subjected to 1·45 tons, which is approximately equal to 2·1 tons force in the member. In the previous example, this member was assumed as a $2\frac{1}{4}'' \times 2\frac{1}{4}'' \times \frac{1}{4}''$ angle with $2-\frac{5}{8}''$ rivets at each end, which will carry a safe load of 2·9 tons over its length.

Design of a Welded Plate Girder.

Data.

Outline of girder	Fig. 14
Class of crane	2
Safe working load	20 tons
Span	50′-0″
Impact factor	1·35
Weight of crab	5·0 tons
,, ,, main girder	3·0 ,,
,, ,, auxiliary girders	0·9 ,,
,, ,, plan bracing	0·7 ,,
,, ,, shafting and bearing	0·7 ,,
,, ,, travel motor	0·6 ,,
,, ,, platform	0·5 ,,
,, ,, handrails	0·05 ,,
Depth of plate girder	3′-6″
Effective width	4′-3″
Wheel base of crab	5′-0″

Distributed Dead Loads.

The distributed dead loads are carried by the main and auxiliary as follows :—

	Main Girder	*Auxiliary Girder*
Main girder	3·0	
Auxiliary girder		0·9
Plan bracing	0·35	0·35
Shafts and bearings	0·35	0·35
Platform	0·25	0·25
Handrails		0·05
	3·95 tons	1·9 tons

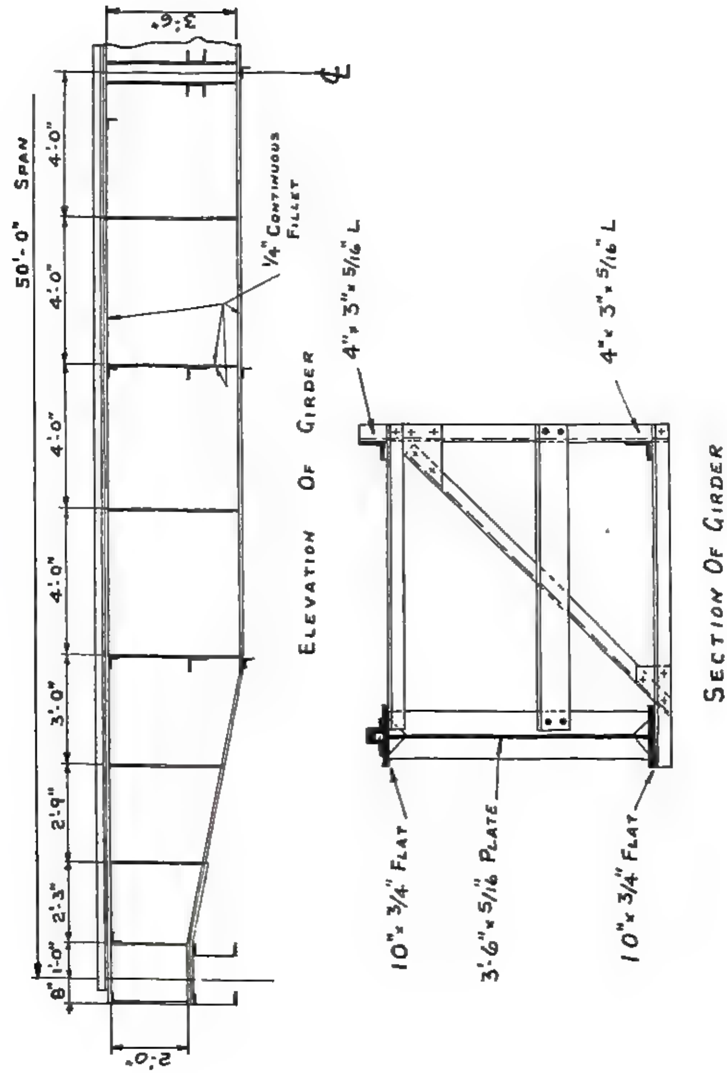

3'-6"
50'-0" Span
4'-0"
4'-0"
4'-0"
4'-0"
3'-0"
2'-9"
2'-3"
1'-0"
8"
2'-0"
1/4" Continuous Fillet
Elevation Of Girder
4" x 3" x 5/16" L
4" x 3" x 5/16" L
10" x 3/4" Flat
3'-6" x 5/16" Plate
10" x 3/4" Flat
Section Of Girder

Fig. 14.

Bending moment (main girder) $= \dfrac{3{\cdot}95 \times 50}{8} = 24{\cdot}7$ tons ft.

Dead Point Load.

The travel motor is a point load acting between both girders equally.

Bending moment (main girder) $= \dfrac{0{\cdot}3 \times 50}{4} = 3{\cdot}75$ tons ft.

Live Load.

Unless it is intended to curtail the flange plate area, it is only necessary to find the maximum bending moment. This will occur with a moving live load when the centre line of the span lies equally spaced between the heaviest loaded crab wheel and the centre of gravity of the wheels. The maximum bending moment occurring at the heaviest loaded wheel as set out in Fig. 15.

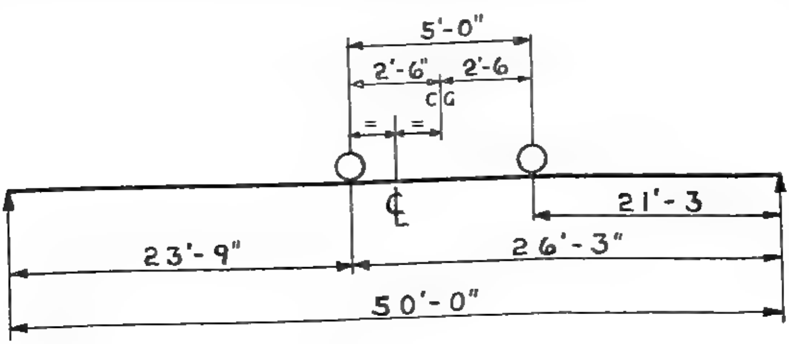


Fig. 15.

Load, including impact, $20 \times 1{\cdot}35$	=	27·0 tons
Crab		5·0 ,,
		32·0 ,,

Wheel load = 8·0 tons.

Left-hand reaction $= \dfrac{8\,(26\frac{1}{4} \times 21\frac{1}{4})}{50} = 7{\cdot}6$ tons.

Bending moment $= 7{\cdot}6 \times 23{\cdot}75 = 181$ tons ft.

A quicker method is by using the expression, which can be easily proved. Maximum B.M. $= \dfrac{W}{2S}\left(S - \dfrac{a}{2}\right)^2$

Where W = wheel load.
S = span.
a = wheel base.

Lateral Loads.

The lateral loads will again be taken as 1/20 of the vertical load.

B.M. distributed dead load $= \dfrac{5{\cdot}85 \times 50}{20 \times 8} = 1{\cdot}82$ tons ft.

i.e., 0·91 tons ft. per flange.

B.M. point dead load $= \dfrac{0{\cdot}6 \times 50}{20 \times 4} = 0{\cdot}36$ tons ft.

i.e., 0·18 tons ft. per flange.

B.M. live load $= \dfrac{180}{20} = 9{\cdot}0$ tons ft.

i.e., 4·5 tons ft. per flange.

The crane is considered to be indoors, therefore wind loads will not be considered.

Web Plate.

The depth of a single web plate crane girder should be approximately one-fifteenth of the span, *i.e.*, in this example 3′-4″.

It is usual to round this figure off to the nearest 3″ above, therefore depth will be taken as 3′-6″.

The thickness will be assumed as $\frac{5}{16}$″.

Ratio of thickness to depth $= \dfrac{42 \times 16}{5} = 135.$

Clause 12 of B.S. 2573 states that if this ratio does not exceed 85 stiffeners are not required, and that this ratio should not exceed 200 for stiffened webs.

Therefore this web thickness is in order if stiffeners are fitted, the pitch of the stiffeners being governed by the following conditions.

The greater unsupported clear dimension of the web in the panel should not exceed $270t$, whilst the lesser unsupported clear dimension of the web in the same panel should not exceed $180t$.

Where t = thickness of web.

The pitch of the stiffeners should also not exceed $1\frac{1}{2}$ times the depth of the web. Thus the latter fixes the pitch, which over the parallel portion of the girder could be 4′-9″ with a lesser pitch over the sloping ends.

The stiffeners also support the long travel shaft bearing brackets and as these are required at 8′-0″ centres, the final pitch of stiffeners will be as Fig. 14.

The shear forces in the girder are set out in Fig. 16 constructed as detailed in the design of the lattice girder.

Each panel of the web should be checked until the parallel portion of the girder is reached.

Panel A.

Shear 17·4 tons.

Depth of web 24 ins.

Area of web $\dfrac{24 \times 5}{16} = 7{\cdot}5$ sq. in.

Shear stress $\dfrac{17{\cdot}4}{7{\cdot}5} = 2{\cdot}3$ tons per sq. in.

As stated in a previous chapter the allowable shear stress is obtained from the following formula

$$6\left[1{\cdot}3 - \frac{\dfrac{b}{t}}{250\left[1+\frac{1}{2}\left(\dfrac{b}{a}\right)^2\right]}\right] \times \text{stress factor.}$$

or 6 tons per sq. in., whichever is the lesser.

a = the greater unsupported clear dimension of web in a panel = 24 ins.

b = the lesser unsupported clear dimension of web in a panel = 20 ins.

t = thickness of web = $\frac{5}{16}$″.

From which the allowable shear is 6·0 tons per sq. in. × stress factor of 0·8 = 4·8 tons per sq. in. Checking the remainder of the panels in the same way.

	Panels			
	B	C	D	E
Shear, tons	17·2	16·1	15·0	13·9
a, ins.	29	35	42	48
b, ins.	27	33	36	42
t, ins.	$\frac{5}{16}$	$\frac{5}{16}$	$\frac{5}{16}$	$\frac{5}{16}$
Area of web sq. in.	8·3	10·0	12·0	13·1
Shear stress tons per sq. in. ...	2·07	1·6	1·25	1·06
Allowable shear stress tons per sq. in.	4·8	4·8	4·7	4·37

The web is understressed, but it is not advisable to reduce below $\frac{5}{16}$″ thick.

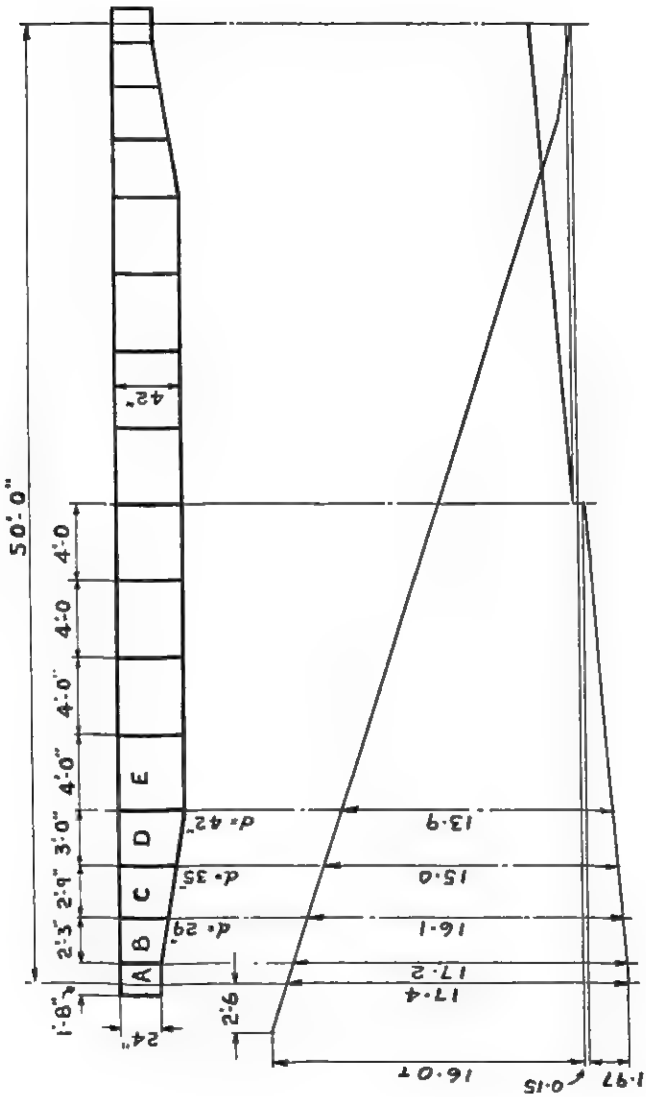

50' 0"
4'0
4'0
4'0"
4'0"
3'0"
2'9"
2'3"
1'8"
2'6
24"
42"
A
B
C
D
E
d=42"
d=35"
d=29"
13·9
15·0
16·1
17·2
17·4
16·0 T
0·15
1·97

Fig. 16.

Top Flange of Main Girder.

Total vertical B.M. $= 24 \cdot 7 + 3 \cdot 75 + 181 = 209 \cdot 45$ tons ft.

$$\text{Flange load from vertical B.M.} = \frac{209 \cdot 45}{3 \cdot 37} = 62 \cdot 2 \text{ tons.}$$

The effective depth of 3·37′ is assumed as the approximate centres of the centres of gravity of the flange areas.

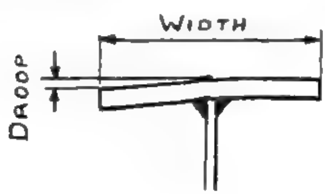
Total lateral B.M. $= 0 \cdot 91 + 0 \cdot 18 + 4 \cdot 5 = 5 \cdot 59$ tons ft.

$$\text{Flange load from lateral B.M.} = \frac{5 \cdot 59}{4 \cdot 25} = 1 \cdot 31 \text{ tons.}$$

Total flange load = 63·5 tons.

In the flange area of a welded construction girder, 1/6 of the web plate area may be taken as constituting part of the flange. However, when this is used, any web plate joint should be designed to transmit both the bending and the shearing stresses. When the flange plate is welded to the web, the edge of the flange plate droops slightly towards the weld.

It can be seen from Fig. 17 that the greater the ratio of width to thickness of the flange plate, the larger the droop.



Width Ins.	Thickness Ins.			
	3/8	1/2	3/4	1
	Droop Sixty Fourths Of An Inch.			
6	3	2	1½	1
9	6	5	3	2
12	9	7	4	3
15	12	9	6	4

For 1/4" Fillets As Above

" 3/8" " Increase 17%

Fig. 17.

The auxiliary girder and the plan bracing will be of riveted construction in order to reduce fatigue stresses. The plan bracing will be connected to the flange plate by 2 rivets, therefore the top flange plate needs to be at least 10″ wide to accommodate same.

The maximum basic bending stress for plate girders is 9·5 tons per sq. in.

This stress is subject to reduction due to flange thickness and width ratio and slenderness ratio. However, with girders of this type, in which the flange plate is braced at fairly close centres to an auxiliary girder, it is most likely the maximum stress is allowable, assuming this

$$\text{Approx. flange area} = \frac{63{\cdot}5}{9{\cdot}5 \times 0{\cdot}8} = 8{\cdot}35 \text{ sq. in.}$$

This allows for a stress factor of Class 2 cranes of 0·8.

The top flange is also subject to a local lateral stress, caused by the lateral wheel load from the crab acting between the centres of plan bracing connections, *i.e.*, 8′-0″.

Lateral wheel load = 1/20 of safe work load + crab weight.

$$= \frac{20 + 5}{20 \times 4} = 0{\cdot}3 \text{ tons per wheel.}$$

Assuming bending moment = WL/6 for part fixity in a continuous flange,

$$\text{Lateral B.M.} = \frac{0{\cdot}3 \times 96}{6} = 4{\cdot}8 \text{ tons in.}$$

Assuming flange plate to be $10'' \times \frac{3}{4}''$.

Flange area

1/6 area of web plate	$\frac{42 \times 5}{6 \times 16}$	= 2·18 sq. in.
Area of flange plate	$10 \times \frac{3}{4}$	= 7·5 „ „
Total flange area		= 9·68 „ „
Compressive stress	$= \frac{63{\cdot}5}{9{\cdot}68}$	= 6·55 tons per sq. in.

The modulus of the top flange about the Y-Y axis

$$= \frac{bd^2}{6} = \frac{0{\cdot}75 \times 10^2}{6} = 12{\cdot}5 \text{ inch}^3 \text{ units.}$$

$$\text{Lateral bending stress} = \frac{4{\cdot}8}{12{\cdot}5} = 0{\cdot}38 \text{ tons per sq, in.}$$

Total compression stress = 6·55 + 0·38 = 6·93 tons per sq. in.

Ratio $d/t = 13{\cdot}3$

Bay length $= 96''$

$k_{yy} = 2{\cdot}14''$

$$l/k_{yy} = \frac{96}{2{\cdot}14} = 45$$

Effective $l/k_{yy} = 45 \times 0{\cdot}85 = 38.$

Reading these values in Table 2 of the Appendix, it will be seen the maximum allowable basis stress of 9·5 tons per sq. in. can be used.

Therefore allowable stress $= 9{\cdot}5 \times$ stress factor $(0{\cdot}8)$.
$= 7{\cdot}6$ tons per sq. in.

Bottom Flange of Main Girder.

This is the tensile flange and the allowable stress is 9·5 tons per sq. in. × a stress factor of 0·8 = 7·6 tons per sq. in.

1/6 area of web	2·18
Area of flange plate $10'' \times \frac{3}{4}''$	7·5
Gross area	9·68 sq. in.
Less $2 - \frac{11}{16}$ rivet holes in $\frac{3}{4}''$ plate	0·86
Nett area	8·82 sq. in.

$$\text{Tensile stress} = \frac{63{\cdot}5}{8{\cdot}82} = 7{\cdot}2 \text{ tons per sq. in.}$$

In this example the top and bottom flanges are of equal section and no further calculation is required. However, if the sections differ, the stresses should be checked on the basis of the moment of inertia.

Stiffeners.

The stiffeners should be proportioned as struts as follows :—

Effective length $= 0{\cdot}75 \times$ depth of web.

The thickness should be at least one sixteenth of the length of the outstanding leg.

The outstanding leg should be at least 2″ plus 1/30 of depth of web.

The load carried by a stiffener is $S = \frac{1}{4}\, V\, P/D$.

Where S = vertical shear on stiffener.
V = maximum vertical shear at stiffener.
D = overall depth of the girder.
P = the sum of the distances between the centre line of the stiffener in question and the centre lines of each of the two adjacent stiffeners.

The total effective section of the stiffener consists of the pair of stiffeners, together with a length of the web on each side of the centre line of the stiffener equal, when available, to twenty times the web thickness.

The radius of gyration should be calculated about the axis parallel to web.

The section of the intermediate stiffeners will be $3\frac{1}{2}'' \times \frac{5}{16}''$ flats as shown in Fig. 18.

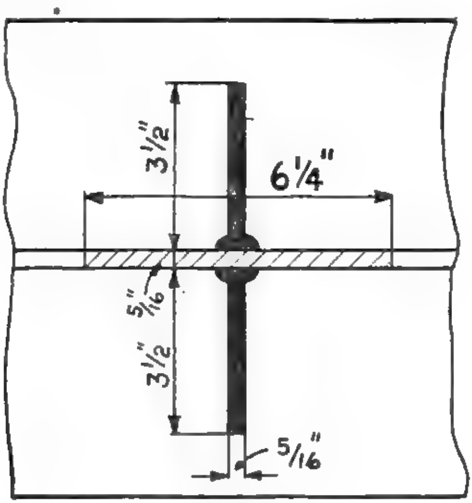


Fig. 18.

$$\begin{aligned}
\text{Area} &= 2 \text{ flats } 3\tfrac{1}{2}'' \times \tfrac{5}{16}'' &&= 2{\cdot}19 \\
&\quad \text{Web } 20 \times \tfrac{5}{16}'' \times \tfrac{5}{16}'' &&= 1{\cdot}95 \\
& && \overline{4{\cdot}14} \text{ sq. in.} \\
\text{Radius of gyration} & && 1{\cdot}57'' \\
\text{Effective length} &= 42 \times 0{\cdot}75 &&= 31{\cdot}5''
\end{aligned}$$

$$\text{Slenderness ratio } l/k_y = \frac{31{\cdot}5}{1{\cdot}57} = 20{\cdot}0$$

$$\text{V} = 13{\cdot}9 \text{ tons.}$$

$$\text{S} \doteq \tfrac{1}{4} \times 13{\cdot}9 \times \frac{(36+48)}{42} = 7{\cdot}0 \text{ tons.}$$

$$\text{Stress} \quad \frac{7{\cdot}0}{4{\cdot}14} = 1{\cdot}7 \text{ tons per sq. in.}$$

Allowable stress (see Table 1 in Appendix) = 6·75 tons per sq. in.

End stiffeners should be proportioned as the intermediate ones but they should extend nearly to the full width of the flange plate.

Section to be used—$4\frac{1}{2}'' \times \frac{3}{8}''$ flats.

Weld Stress.

The basic stress in fillet welds is 6·5 tons per sq. in for a thickness equal to the throat thickness.

For Class 2 cranes the stress will be 6·5 × stress factor (0·8) = 5·2 tons per sq. in.

The prepared butt weld is considered as equal to the full strength of the parent metal.

Flange Welds.

The weld between the top flange plate and web plate carries the horizontal shear and the vertical shear of the wheel load.

The wheel load is considered as spread over a length equal to three times the depth from the rail head to the line of weld.

The weld between the bottom flange plate and the web plate carries the horizontal shear only.

Investigations made by various technical bodies have found that longitudinal welds along a flange should be continuous in order to reduce fatigue points to a minimum.

Vertical load from crab wheel 8·0 tons.

Spread of wheel load $= 3\,(2\frac{5}{8}'' + \frac{3}{4}'') = 10\frac{1}{8}''$

Vertical load per inch $= \dfrac{8 \cdot 0}{10 \cdot 125} = 0 \cdot 79$ tons per in.

Maximum vertical shear at girder end = 17·4 tons.

Horizontal shear per inch carried by weld $= \dfrac{V}{d} \times \dfrac{A_1}{A_1 + A_2}$

Where V = vertical shear (tons).
d = depth between flange welds (inches).
A_1 = area of flange plate.
A_2 = area of web allowed in flange.

Horizontal shear in weld per inch in end panel $= \dfrac{17 \cdot 4}{24} \times \dfrac{7 \cdot 5}{7 \cdot 5 + 2 \cdot 18}$

$= 0 \cdot 57$ tons per in.

The resultant shear from the vertical and horizontal loads $= \sqrt{0 \cdot 79^2 + 0 \cdot 57^2} = 0 \cdot 97$ tons per in.

Safe load of 1″ of ¼″ fillet weld

$$= \frac{1}{\sqrt{2}} \times 0{\cdot}25 \times 5{\cdot}2 \text{ tons} = 0{\cdot}91 \text{ tons.}$$

This load in the flange fillets is a fluctuating one varying from approximately 25% full load to 100% full load, and in designs such as crane girders, where the loading is dynamic, the safe load of welds should be reduced by multiplying by the factor k as shown in the following table :—

$\dfrac{f \text{ min.}}{f \text{ max.}^{m}}$	k
+1·0	1·0
+0·5	1·0
0	0·56

Where f max. = maximum stress.
f min. = minimum stress.

Interpolating for the conditions of this design $k = 0{\cdot}78$.

Therefore safe load on 2 - ¼″ fillet welds

$= 2 \times 0{\cdot}91 \times 0{\cdot}78$

$= 1{\cdot}42$ tons

The bottom flange welds will also be 2 - ¼″ fillet welds ; this only carries the horizontal shear of 0·64 tons per in.

Stiffener Welds.

There are many varying views on stiffener welds, whether continuous or intermittant welds should be used and whether the ends of the stiffeners should be welded to the flange plate.

After investigating all these views and receiving expert advice from leading welding companies, the following points are recommended.

(1) The stiffener should be welded to the web plate by two continuous welds.

(2) Neither longitudinal nor transverse welds should be used between the end of the stiffener and either the tensile or compressive flange plate.

In many plate girder designs not for cranes, it has been a practice in the past to weld the stiffener to the compression flange by transverse welds, and to weld the tension flange end of the stiffener to a small pad, this pad being welded to the flange plate by longitudinal welds.

However, in crane structures, which have a large variation of stress, no weld is recommended. The stiffener should be cut to exact length equal to the web depth and welded to the web plate first, thus when the flange plate is welded to the web, the tendency of the flange plate to droop will make a tight fit between flange and stiffener.

Outrigger.

The auxiliary girder and plan bracing should be designed on similar lines to that shown in the previous design of the lattice girder.

Detailing.

The joints in the web plate and the flange plates should not be in line but both should be made by prepared butt welds.

Stiffeners should be fitted in pairs to reduce unbalanced shrinkage stresses and consequent distortion.

The flange plate should consist of one plate only and if the design requires a curtailment of area, it is most satisfactory to maintain the thickness of the plate and reduce the width with a gradual slope and thus obtain a good stress flow.

The welding of the girder will shorten the resultant structure by approximately $\frac{1}{16}''$ for every 20′-0″ in length. An allowance should be made for this when calling for the plates.

The camber in the girder can be obtained by the sequence of welding, cutting the girder ends and the welding of the web butt joints.

Design of a Riveted Plate Girder.

This example is to be used as a comparison with the previous design and therefore the same data, bending moments and shear will be used.

An outline of the girder is shown in Fig. 19.

Class of crane		2
Safe working load		20 tons
Span		50′-0″
Impact factor		1·35
Weight of crab		5·0 tons
,, ,, main girder		3·0 tons
,, ,, auxiliary girder		0·9 tons
,, ,, plan bracing		0·7 tons
,, ,, shafting and bearing		0·7 tons
,, ,, travel motor		0·6 tons
,, ,, platform		0·5 tons
,, ,, handrails		0·05 tons
Depth of plate girder		3′-6″
Effective width		4′-3″
Wheelbase of crab		5′-0″
Total vertical bending moment	=	209·45 tons ft.
Total lateral bending moment	=	5·59 ,, ,,
Total flange load	=	63·5 tons
Local lateral bending moment	=	4·8 tons ins.

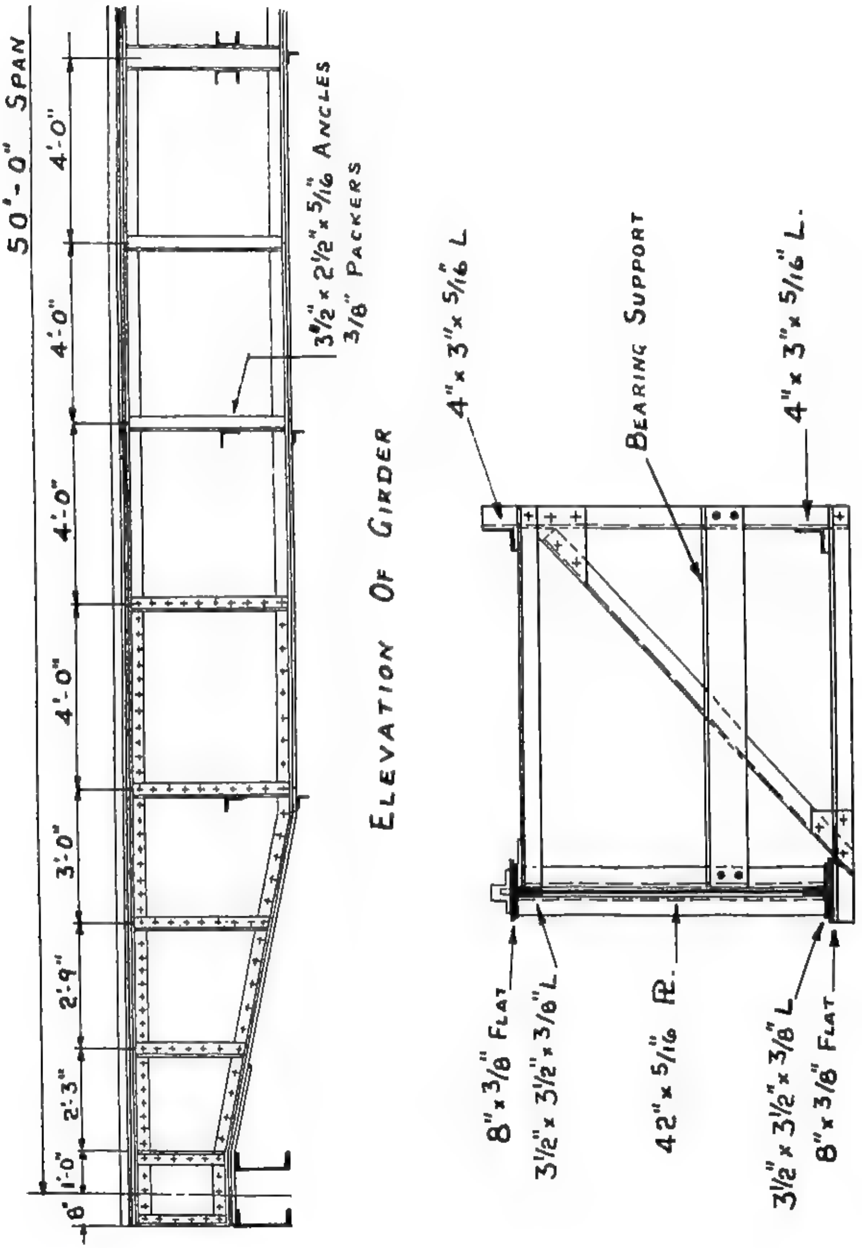

50'-0" Span
4'-0"
4'-0"
4'-0"
4'-0"
3'-0"
2'-9"
2'-3"
1'-0"
8"
3½" x 2½" x 5/16" Angles
3/8" Packers
Elevation Of Girder
4" x 3" x 5/16" L
Bearing Support
4" x 3" x 5/16" L.
8" x 3/8" Flat
3½" x 3½" x 3/8" L
42" x 5/16" Pl.
3½" x 3½" x 3/8" L
8" x 3/8" Flat

Fig. 19.

The web plate design is exactly as in the welded girder, thus the size will be 3′-6″ × $\frac{5}{16}$″.

In the flange area of a riveted construction girder, $\frac{1}{8}$ of the web plate area may be taken as constituting part of the flange, if any web joint is designed to transmit both the bending and the shearing stresses.

Top Flange.

$\frac{1}{8}$ area of web plate	$= \frac{36}{8} \times \frac{5}{16}$	=	1·64 sq. in.
Assume flange plate	$8'' \times \frac{3}{8}$	=	3·0 ,, ,,
Assume flange angles	$2\text{-}3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$	=	4·98 ,, ,,
Total area			9·62 ,, ,,

$$t_e = \frac{\text{Area of horizontal portion of flange}}{\text{Width of flange}}$$

$$= \frac{3{\cdot}0 + (2 \times 3\frac{1}{2} \times \frac{3}{8})}{8} = 0{\cdot}7 \text{ ins.}$$

Depth of Web = 42″

Ratio d/t_e $= \frac{42}{0{\cdot}7} = 60$

Bay length = 96″

Radius of gyration about YY axis (k_{yy}) = 1·4″

Slenderness ratio $= \frac{96}{1{\cdot}4} = 69$

Effective slenderness ratio = 69 × 0·85 = 51

Modulus of top flange about YY axis = 7·1 inch^3 units.

Safe compressive stress 0·8 × 9·5 = 7·6 tons per sq. in.

Compressive stress $= \frac{63{\cdot}5}{9{\cdot}62}$ = 6·6

Local lateral stress $= \frac{4{\cdot}8}{7{\cdot}1}$ = 0·7

Total compressive stress = 7·3 tons per sq. in.

Bottom Flange.

Using the same section as the top flange

$\frac{1}{8}$ area of web plate		1·64
Flange plate	$8'' \times \frac{3}{8}''$	3·0
Flange angles	$2\text{-}3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$	4·98
Gross area		9·62 sq. in.
Less $2\text{-}\frac{13}{16}''$ holes in $\frac{3}{4}''$ thick member		1·2
Nett area		8·42 sq. in.

$$\text{Tensile stress} = \frac{63{\cdot}5}{8{\cdot}42} = 7{\cdot}55 \text{ tons per sq, in.}$$

$$\text{Allowable stress} = 7{\cdot}6 \text{ tons per sq. in.}$$

Stiffeners.

The proportional requirements for the stiffeners are exactly as stated in the welded design. To satisfy these the section of the stiffeners will be $2\text{-}3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$ angles with the $3\frac{1}{2}''$ legs outstanding.

Again an area of web equal to 20 times the web thickness can be included in the stiffener area.

Flange Rivets.

The rivets between the flange angles and the web plate of the top flange transmit both the horizontal shear and the vertical shear from the wheel load.

The wheel load is considered as spread over a length equal to three times the depth from the rail head to the centre line of the rivet group in the web, *i.e.*, $3\ (2\frac{5}{8}'' + \frac{3}{8}'' + 2'') = 15$ ins.

$$\therefore \text{ Vertical shear per inch} = \frac{\text{Wheel load}}{15} = \frac{8}{15} = 0{\cdot}53 \text{ tons per in.}$$

Horizontal shear per in. (Rivets in web plate) $= \dfrac{V}{d} \times \dfrac{A_1 + A_2}{A_1 + A_2 + A_3}$

Where V = vertical shear.
d = effective depth.
A_1 = area of flange plates.
A_2 = area of flange angles.
A_3 = area of web plate allowed in flange.

Vertical shear at ends = 17·4 tons
Depth at ends = 24 ins.

Horizontal shear per in. in rivets between flange angles and web plate $= \dfrac{17{\cdot}4}{24} \times \dfrac{3{\cdot}0 + 4{\cdot}98}{3{\cdot}0 + 4{\cdot}98 + 1{\cdot}64}$

$= 0{\cdot}6$ tons per in.

Resultant shear from vertical loads and horizontal loads

$= \sqrt{0{\cdot}53^2 + 0{\cdot}6^2} = 0{\cdot}8$ tons.

The safe load of one $\frac{3}{4}''$ dia. rivet bearing in a $\frac{5}{16}''$ web plate $= 2{\cdot}81$ tons (Table 3).

$$\therefore \quad \text{Pitch of rivets} = \frac{2{\cdot}81}{0{\cdot}8} = 3{\cdot}5 \text{ inches.}$$

As the shear reduces and the depth of the girder increases, the rivet pitch can be increased.

The maximum pitch in the compression flange is 12 times the thickness of the thinnest outside plate.

The rivets attaching the flange plate to the angles transmit only horizontal shear. The following gives the value of this shear.

$$= \frac{V}{d} \times \frac{A_1}{A_1 + A_2 + A_3}$$

$$= \frac{17{\cdot}4}{24} \times \frac{3{\cdot}0}{3{\cdot}0 + 4{\cdot}98 + 1{\cdot}64} = 0{\cdot}226 \text{ tons per in.}$$

These rivets are placed in pairs, thus the strength of

2-$\frac{5}{8}''$ rivets in $\frac{3}{8}''$ bearing $=$ 5·6 tons.
2-$\frac{5}{8}''$ rivets in single shear $=$ 3·2 tons.

$\frac{5}{8}''$ dia. rivets being used as these rivets also pass through the rail and it is difficult to close larger than $\frac{5}{8}''$ rivets in a 56-lbs. per yard rail, which is a size suitable for this crane.

Some manufacturers prefer to attach the top plate to the girder with C.S.K. rivets and rivet the rail on afterwards with a rivet pitch of approximately 2′-0″.

$$\text{Pitch of rivets} = \frac{3{\cdot}2}{0{\cdot}226} = 14 \text{ ins.}$$

This pitch should be kept at a maximum of 6″.

The rivet pitch in the bottom flange plate and angles is calculated as for the top flange but omitting the wheel load shear.

The maximum rivet pitch in the tension flange should not exceed 16 times the thickness of the thinnest outside plate or member.

For comparison the following are the weights of the riveted and welded main girders :—

Riveted	3·04 tons.
Welded	2·81 tons.

Thus by welding we gain a weight saving of $7\frac{1}{2}\%$ over riveted design, but the cost per ton generally is greater for welding. This cost varies considerably with manufacturers and depends largely on available plant and experience in the type of work.

Design of a R.S.J. Girder.

Data.

Class of crane	2
Safe working load	10 tons
Span	20′-0″
Impact factor	1·35
Weight of crab	3·0 tons
,, ,, one main girder	0·9 ,,
,, ,, auxiliary girder	0·3 ,,
,, ,, plan bracing	0·3 ,,
,, ,, shafting and bearings	0·3 ,,
,, ,, long travel motor	0·4 ,,
,, ,, platform	0·2 ,,
Wheel base of crab	4′-0″

In designs of R.S.J. type girders it is normal to standardise sizes of sections and the conditions of loading have to be generalised.

The platform and bracing may be attached to the top flange, bottom flange or in an intermediate position (see Fig. 20), this is to suit site headroom conditions.

The ungeared girder has no outrigger; thus it must carry any horizontal loads unaided. Therefore it is generally accepted to design these girders to carry the vertical and horizontal loads without assistance from plan bracing, unless circumstances call for a special design.

Distributed vertical load carried by main girder.

Main girder weight	0·9 tons
Part plan bracing	0·15 ,,
Part shafting and bearing	0·15 ,,
Part platform	0·1 ,,
	1·4 ,,

$$\text{Bending moment} = \frac{1{\cdot}4 \times 20 \times 12}{8} = 42 \text{ tons ins.}$$

Point vertical load carried by main girder at centre of span.

Part long travel motor weight = 0·2 tons.

$$\text{Bending moment} = \frac{0{\cdot}2 \times 20 \times 12}{4} = 12 \text{ tons ins.}$$

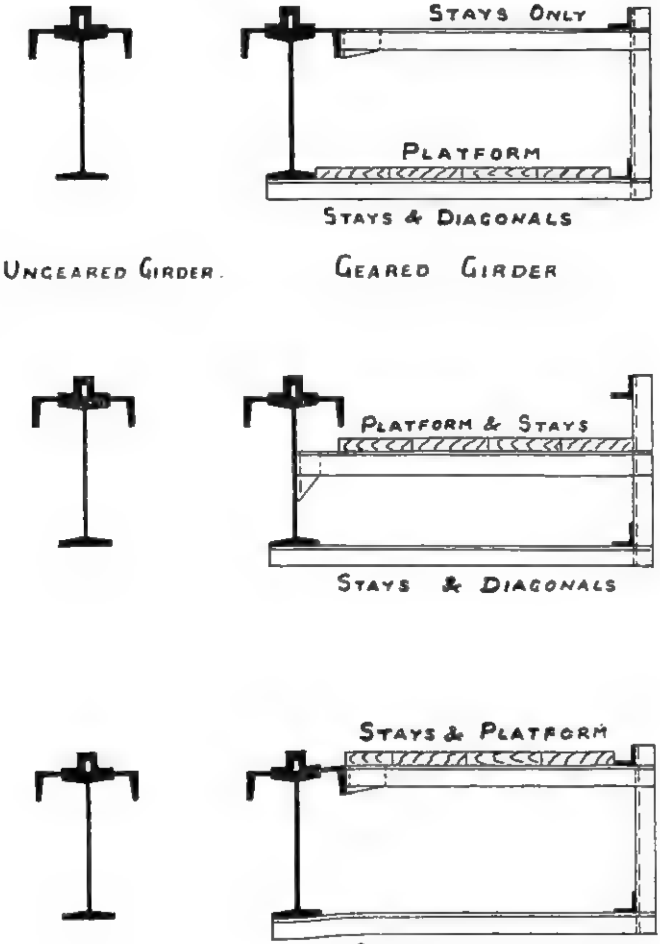

STAYS ONLY
PLATFORM
STAYS & DIAGONALS
UNGEARED GIRDER.
GEARED GIRDER
PLATFORM & STAYS
STAYS & DIAGONALS
STAYS & PLATFORM
STAYS & DIAGONALS

Fig. 20.

The live load is positioned to give maximum bending moment as stated in the design of the welded girder.

Safe working load and impact	$= 10 \times 1{\cdot}35$	=	13·5 tons
Crab weight		=	3·0 ,,
			16·5 ,,

The wheel load is assumed to be equally distributed over all four wheels.

$$\therefore \quad \text{Wheel load} = \frac{16{\cdot}5}{4} = 4{\cdot}125 \text{ tons.}$$

Referring to Fig. 21.

$$\text{Left-hand reaction} = \frac{4{\cdot}125 \times (7+11)}{20} = 3{\cdot}71 \text{ tons.}$$

Maximum bending moment $= 3{\cdot}71 \times 9 \times 12 = 3{\cdot}98$ tons ins.
Total vertical bending moment $= 42 + 12 + 3{\cdot}98$
$= 452$ tons ins.

Distributed lateral dead load bending moment.

Main girder	0·9 tons
Auxiliary girder	0·3 ,,
Plan bracing	0·3 ,,
Shafting and bearing	0·3 ,,
Platform	0·2 ,,
	2·0 ,,

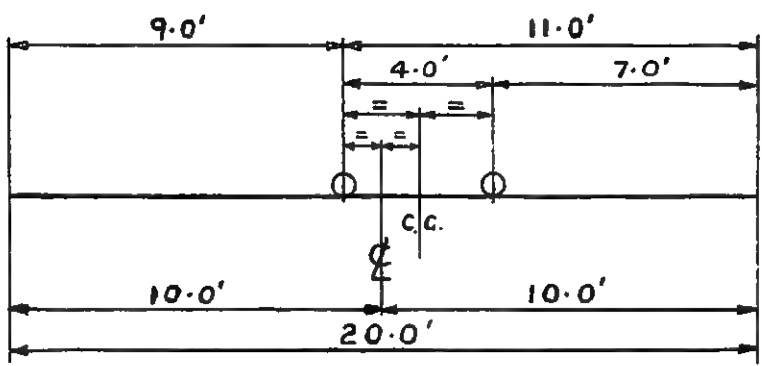


Fig. 21.

Bending moment $\frac{1}{20} \times \frac{2{\cdot}0 \times 20 \times 12}{8} = 3{\cdot}5$ tons ins.

Point lateral load bending moment.

Long travel motor $= 0{\cdot}4$ tons.

Bending moment $\frac{1}{20} \times \frac{0{\cdot}4 \times 20 \times 12}{4} = 1{\cdot}2$ tons ins.

These two lateral bending moments from dead loads are distributed between each flange equally.

$\therefore$ Lateral bending moment carried by each flange

$$= \frac{1{\cdot}2 + 3{\cdot}5}{2} = 2{\cdot}35 \text{ tons ins.}$$

Lateral live load bending moment.

Safe working load	10 tons
Crab weight	3 ,,
	13 ,,

By proportion to the vertical bending moment.

$$\text{Lateral live bending moment} = \frac{1}{20} \times 398 \times \frac{13}{16{\cdot}5}$$

$$= 15{\cdot}8 \text{ tons ins.}$$

This bending moment is applied to the top flange only.

Section	$18'' \times 7''$ R.S.J.
Properties of section.	
Inertia	1151 inch^4 units.
Modulus Z_{xx}	127·9 inch^3 units.
Modulus Z_{yy}	6·65 inch^3 units per flange.
d =	18 inches.
K_1 =	1·0
K_2 =	0
t_e =	0·9″
d/t_e =	20

Vertical compressive stress $= \frac{452}{127{\cdot}9} = 3{\cdot}53$ tons per sq. in.

Lateral compressive stress $= \frac{15{\cdot}8 + 2{\cdot}35}{6{\cdot}65} = 2{\cdot}73$ tons per sq. in.

Total compressive stress $= 6{\cdot}26$ tons per sq. in.

Unsupported length $= 240$ ins.

$l/k_{yy} = 166$

For a normal R.S.J. type of girder with large gusset plates to the end carriages the effective length will be considered as 0·85 of the crane span.

Effective $l/k_{yy} = 166 \times 0{\cdot}85 = 141$.

From this and using the tables to obtain values of A and B, the compressive stress is limited to 10 tons per sq. in. × the stress factor (0·8 for Class 2) = 8 tons per sq. in.

Vertical tensile stress $= \dfrac{452}{127{\cdot}9} = 3{\cdot}53$ tons per sq. in.

Lateral tensile stress $= \dfrac{2{\cdot}35}{6{\cdot}65} = 0{\cdot}35$ „ „ „ „

Total tensile stress $= 3{\cdot}88$ „ „ „ „

Allowable tensile stress $= 10 \times 0{\cdot}8 = 8$ „ „ „ „

From the above it will be seen that the 18″ × 7″ R.S.J. is very lowly stressed, and in order to obtain a fully stressed section a compounded R.S.J. would have to be used.

However, the additional manufacturing costs of a compounded section outweigh the saving in weight.

TABLE 1.

COMPRESSIVE STRESSES IN AXIALLY LOADED STRUTS.

l/k	Allowable Stress Tons per Sq. In.					
	Basic	Class of Crane				
		0	1	2	3	4
0	9·00	8·10	7·65	7·20	6·75	6·30
10	8·72	7·85	7·41	6·97	6·54	6·10
20	8·44	7·58	7·17	6·75	6·33	5·91
30	8·15	7·34	6·93	6·52	6·11	5·71
40	7·82	7·04	6·65	6·25	5·87	5·47
50	7·48	6·73	6·36	5·98	5·61	5·24
60	7·04	6·34	5·98	5·63	5·28	4·93
70	6·55	5·89	5·57	5·24	4·91	4·59
80	6·02	5·42	5·12	4·82	4·51	4·21
90	5·44	4·90	4·62	4·35	4·08	3·81
100	4·86	4·37	4·13	3·89	3·65	3·40
110	4·32	3·89	3·67	3·46	3·24	3·02
120	3·84	3·46	3·26	3·07	2·88	2·69
130	3·40	3·06	2·89	2·72	2·55	2·38
140	3·02	2·72	2·57	2·42	2·27	2·11
150	2·71	2·44	2·30	2·17	2·03	1·90
160	2·42	2·18	2·06	1·94	1·82	1·69
170	2·20	1·98	1·87	1·76	1·65	1·54
180	1·97	1·77	1·67	1·58	1·48	1·38
190	1·80	1·62	1·53	1·44	1·35	1·26
200	1·64	1·48	1·39	1·31	1·23	1·15

l = Effective Length.

k — Radius of Gyration.

TABLE 2.

BENDING STRESSES.

VALUES OF A AND B TONS PER SQUARE INCH.

l/k_Y	Values of A — Ratio of d/t_e																		B	l/k_Y
	10	11	12	13	14	15	16	17	18	19	20	25	30	40	50	60	80	100		
80	32·1	29·8	28·0	26·6	25·4	24·2	23·4	22·7	22·1	21·4	20·9	19·2	18·2	17·1	16·6	16·3	16·0	15·9	15·6	80
90	27·8	25·7	24·1	22·8	21·6	20·6	19·8	19·1	18·5	18·0	17·6	15·9	14·9	13·8	13·3	13·0	12·7	12·6	12·4	90
100	24·4	22·7	21·1	19·9	18·8	18·0	17·2	16·6	16·0	15·5	15·0	13·4	12·5	11·5	11·0	10·7	10·4	10·3	10·0	100
110	21·9	20·2	18·8	17·7	16·7	15·9	15·2	14·6	14·0	13·5	13·1	11·6	10·7	9·7	9·2	8·9	8·7	8·5	8·3	110
120	19·9	18·3	17·0	16·0	15·0	14·2	13·6	13·0	12·5	12·0	11·6	10·2	9·3	8·4	7·9	7·6	7·4	7·2	6·9	120
130	18·2	16·8	15·5	14·5	13·7	12·9	12·3	11·7	11·2	10·8	10·4	9·1	8·2	7·3	6·9	6·6	6·3	6·2	5·9	130
140	16·8	15·2	14·2	13·3	12·5	11·8	11·2	10·7	10·2	9·8	9·5	8·2	7·4	6·5	6·0	5·8	5·5	5·4	5·1	140
150	15·6	14·3	13·2	12·3	11·5	10·9	10·3	9·8	9·4	9·0	8·7	7·4	6·7	5·8	5·4	5·1	4·8	4·7	4·4	150
160	14·5	13·4	12·3	11·5	10·7	10·1	9·6	9·1	8·7	8·3	8·0	6·8	6·1	5·2	4·8	4·5	4·3	4·2	3·9	160
170	13·6	12·4	11·5	10·8	10·0	9·4	8·9	8·5	8·1	7·7	7·4	6·3	5·6	4·8	4·4	4·1	3·9	3·7	3·5	170
180	12·8	11·6	10·8	10·1	9·4	8·8	8·4	7·9	7·5	7·2	6·9	5·9	5·2	4·4	4·0	3·7	3·5	3·3	3·1	180
190	12·1	11·1	10·2	9·5	8·9	8·3	7·9	7·5	7·1	6·8	6·5	5·5	4·8	4·05	3·65	3·45	3·25	3·0	2·8	190
200	11·5	10·4	9·6	8·9	8·3	7·8	7·4	7·0	6·7	6·4	6·1	5·1	4·5	3·8	3·4	3·1	2·9	2·7	2·5	200

TABLE 3.

POWER DRIVEN RIVETS AND FIT BOLTS.

Basic Values **Stress Factor — 1**

Dia. of Rivet or Bolt	6·5 Ton / Sq. In. Single Shear	Double Shear	Bearing Value 15 Ton / Sq. In. Thickness of Member Passed Through 1/4"	5/16"	3/8"	7/16"	1/2"	9/16"	5/8"	11/16"	3/4"
3/8"	0·72	1·44	1·41	1·76							
7/16"	0·98	1·95	1·64	2·05							
1/2"	1·28	2·55	1·88	2·34	2·81						
9/16"	1·62	3·23	2·11	2·64	3·16	3·69					
5/8"	1·99	3·99	2·34	2·93	3·52	4·10					
11/16"	2·41	4·83	2·58	3·22	3·87	4·51	5·16				
3/4"	2·87	5·74	2·81	3·52	4·22	4·92	5·63	6·33			
13/16"	3·37	6·74	3·05	3·81	4·57	5·33	6·09	6·86			
7/8"	3·91	7·82	3·28	4·10	4·92	5·74	6·56	7·38	8·20		
15/16"	4·49	8·97	3·52	4·39	5·27	6·15	7·03	7·91	8·79	9·67	
1"	5·11	10·21	3·75	4·69	5·63	6·56	7·50	8·44	9·38	10·31	
1 1/16"	5·76	11·53	3·98	4·98	5·98	6·97	7·97	8·96	9·96	10·96	11·95

Class 2 **Stress Factor = 0.8.**

Dia. of Rivet or Bolt	5·2 Ton / Sq. In. Single Shear	Double Shear	Bearing Value 12 Ton / Sq. In. Thickness of Member Passed Through. 1/4"	5/16"	3/8"	7/16"	1/2"				
3/8"	0·57	1·15	1·13	1·41							
7/16"	0·78	1·56	1·31	1·64							
1/2"	1·02	2·04	1·50	1·88	2·25						
9/16"	1·29	2·58	1·69	2·11	2·53	2·95					
5/8"	1·60	3·19	1·88	2·34	2·81	3·28					
11/16"	1·93	3·86	2·06	2·58	3·09	3·61	4·13				
3/4"	2·30	4·59	2·25	2·81	3·38	3·94	4·50	5·06			
13/16"	2·70	5·39	2·44	3·05	3·66	4·27	4·88	5·48			
7/8"	3·13	6·25	2·63	3·28	3·94	4·59	5·25	5·91	6·56		
15/16"	3·59	7·18	2·81	3·52	4·22	4·92	5·63	6·33	7·03	7·73	
1"	4·08	8·17	3·00	3·75	4·50	5·25	6·00	6·75	7·50	8·25	
1 1/16"	4·61	9·22	3·19	3·98	4·78	5·58	6·38	7·17	7·97	8·77	9·56

BIBLIOGRAPHY.

B.S. 2573. 1955. *Report of Permissible Stresses in Crane Structures.*

B.S. 449. 1948. *The Use of Structural Steel in Building.*

B.S. 446. *Overhead Electric Cranes.* (At present being revised).

B.I.S.R.A. D. Buchanan, B.Sc. (Eng.) *Crane Specification for Heavy Duty Cranes.* B.I.S.R.A. PE/H/28/55.

G. M. Boyd. *Brittle Fracture Problems in Steel Construction.*

R. Weck. *Fatigue of Welded Fractures.* (*The Structural Engineer*, Vol. 32, No. 4 (Apl., 1954)).

Dr. E. Lightfoot. Dr. N. Jackson. *Static Stresses in Electric Overhead Travelling Crane Girders.* (*The Structural Engineer*, Vol. 35, No. 1 (Jan., 1957)).

R. G. Braithwaite. *Welded Highway Bridges.*

D. J. Davies. Journal of Institution of Civil Engineers. No. 6. 1949-50.

R. G. Tyler. British Iron and Steel Research Association. *The Deflection Method of Design for E.O.T. Cranes* (PE/HE/71/55).

D. Buchanan & R. G. Tyler. British Iron and Steel Research Association. *Structural Design of Steelworks Electric Overhead Travelling Cranes* (PE/H/27/57).

ACKNOWLEDGMENTS.

The author is grateful to the British Standards Institution for permission to quote formulae from the following British Standard : B.S. 2573 (1955). *Report of Permissible Stresses in Crane Structures.*

Copies of this publication may be obtained from the following address (Price 4/6 each) :—

British Standards Institution,
British Standards House,
2 Park Street,
London, W.1.

The author also wishes to thank the British Iron and Steel Research Association for permission to quote from their papers PE/H/27/57, *Structural Design of Steelworks' Overhead Travelling Cranes.*

A.E.S.D. PRINTED PAMPHLETS AND OTHER PUBLICATIONS IN STOCK.

An up-to-date list of A.E.S.D. pamphlets in stock is obtainable on application to the Editor, *The Draughtsman*, Onslow Hall, Little Green, Richmond, Surrey.

A similar list is also published in *The Draughtsman* twice a year.

Readers are asked to consult this list before ordering pamphlets published in previous sessions.

LIST OF A.E.S.D. DATA SHEETS.

1.	Safe Load on Machine-Cut Spur Gears.	
2.	Deflection of Shafts and Beams.	Connected.
3.	Deflection of Shafts and Beams (Instruction Sheet).	
4.	Steam Radiation Heating Chart.	
5.	Horse-Power of Leather Belts, etc.	
6.	Automobile Brakes (Axle Brakes).	Connected.
7.	Automobile Brakes (Transmission Brakes).	
8.	Capacities of Bucket Elevators.	
9.	Valley Angle Chart for Hoppers and Chutes.	
10.	Shafts up to 5½ inch diameter, subjected to Twisting and Combined Bending and Twisting.	
11.	Shafts, 5¾ to 26 inch diameter, subjected to Twisting and Combined Bending and Twisting.	
12.	Ship Derrick Booms.	
13.	Spiral Springs (Diameter of Round or Square Wire).	
14.	Spiral Springs (Compression).	
15.	Automobile Clutches (Cone Clutches).	
16.	,, ,, (Plate Clutches).	
17.	Coil Friction for Belts, etc.	
18.	Internal Expanding Brakes. Self-Balancing Brake Shoes (Force Diagram).	Connected.
19.	Internal Expanding Brakes. Angular Proportions for Self-Balancing.	
20.	Referred Mean Pressure Cut-Off, etc.	
21.	Particulars for Balata Belt Drives.	
22.	⅞" Square Duralumin Tubes as Struts.	
23.	1" ,, ,, ,, ,,	
24.	¾" Square Steel Tubes as Struts (30 ton yield).	
25.	⅞" ,, ,, ,, (30 ton yield).	
26.	1" ,, ,, ,, (30 ton yield).	
27.	¾" ,, ,, ,, (40 ton yield).	
28.	⅞" ,, ,, ,, (40 ton yield).	
29.	1" ,, ,, ,, (40 ton yield).	
30.	Moments of Inertia of Built-up Sections (Tables).	Connected.
31.	Moments of Inertia of Built-up Sections (Instructions and Examples).	
34.	Capacity and Speed Chart for Troughed Band Conveyors.	
35.	Screw Propeller Design (Sheet 1, Diameter Chart).	Connected.
36.	,, ,, ,, (Sheet 2, Pitch Chart).	
37.	,, ,, ,, (Sheet 3, Notes & Examples).	
38.	Open Coil Conical Springs.	
39.	Close Coil Conical Springs.	
40.	Trajectory Described by Belt Conveyors (Revised, 1949).	
41.	Metric Equivalents.	
42.	Useful Conversion Factors.	
43.	Torsion of Non-Circular Shafts.	
44.	Railway Vehicles on Curves.	
46.	Coned Plate Development.	
47.	Solution of Triangles (Sheet 1, Right Angles).	
48.	Solution of Triangles (Sheet 2, Oblique Angles).	
49.	Relation between Length, Linear Movement and Angular Movement of Lever (Diagram and Notes).	
50.	,, ,, ,, ,, ,, ,, (Chart).	
51.	Helix Angle and Efficiency of Screws and Worms.	
52.	Approximate Radius of Gyration of Various Sections.	
53.	Helical Spring Graphs (Round Wire.	Connected.
54.	,, ,, ,, (Round Wire).	
55.	,, ,, ,, (Square Wire).	

56. Relative Value of Welds to Rivets.
58. Graphs for Strength of Rectangular Flat Plates of Uniform Thickness.
59. ,, Deflection ,, ,, ,, ,, ,,
61. Deflection of Leaf Spring.
62. Strength of Leaf Spring.
63. Chart Showing Relationship of Various Hardness Tests.
64. Shaft Horse-Power and Proportions of Worm Gear.
65. Ring with Uniform Internal Load (Tangential Strain). Connected.
66. ,, ,, ,, ,, ,, (Tangential Stress).
67. Hub Pressed on to Steel Shaft. (Maximum Tangential Stress at Bore of Hub).
68. Hub Pressed on to Steel Shaft. (Radial Gripping Pressure between Hub and Shaft).
69. Rotating Disc (Steel) Tangential Strain. Connected.
70. ,, ,, ,, Stress.
71. Ring with Uniform External Load, Tangential Strain. Connected.
72. ,, ,, ,, ,, ,, Stress.
73. Viscosity Temperature Chart for Converting Commercial to Absolute Viscosities. Connected.
74. Journal Friction on Bearings.
75. Ring Oil Bearings.
76. Shearing and Bearing Values for High Tensile Structural Steel Shop Rivets in accordance with B.S.S. No. 548/1934.
78. Velocity of Flow in Pipes for a Given Delivery. Connected.
79. Delivery of Water in Pipes for a Given Head.
80. (See No. 105).
81. Involute Toothed Gearing Chart.
83. Variation of Suction Lift and Temperature for Centrifugal Pumps.
89. Curve Relating Natural Frequency and Deflection.
90. Vibration Transmissibility Curved or Elastic Suspension. Connected.
91. Instructions and Examples in the Use of Data Sheets, Nos. 89 and 90.
92. Pressure on Sides of Bunker.
93-4-5-6-7. Rolled Steel Sections.
98-9-100. Boiler Safety Valves.
102. Pressure Required for Blanking and Piercing.
103. Punch and Die Clearances for Blanking and Piercing.
104. Nomograph for Valley Angles of Hoppers and Chutes.
105. Permissible Working Stresses in Mild Steel Struts with B.S. 449, 1948.
106. Compound Cylinder (Similar Material) Radial Pressure of Common Diameter (D1).
107. True Angles for Pipe Bends
108. Development of Spiral Chutes.
109. Vertical Pitch of Panels in a Frame with Bracing at a Constant Slope.
110 Permissible Stresses in Cranes—B.S. 2573. Table 1—Values of A & B Tons/Sq. In.
111. Permissible Stresses in Cranes—B.S. 2573. Table 2—Values of A & B Tons/Sq. In.
112. Permissible Stresses in Cranes—B.S. 2573. Table 3—Compressive Stresses in Axially Loaded Struts.
113. Permissible Stresses in Cranes—B.S. 2573. Table 4—Power Driven Rivets and Fit Bolts.
114. Permissible Stresses in Cranes—B.S. 2573. Table 5—Power Driven Rivets and Fit Bolts.
115. Rectangular Weld Groups with Eccentric Load in Plane of Weld.

(**Data Sheets are 3d to Members, 6d to others, post free**).

Orders for Pamphlets and Data Sheets to be sent to the Editor, *The Draughtsman*, cheques and orders being crossed "A.E.S.D."

ADVICE TO INTENDING AUTHORS OF A.E.S.D. PRINTED PAMPHLETS.

Pamphlets submitted to the National Technical Sub-Committee for consideration with a view to publication in this series should not exceed 10,000 to 15,000 words and about 20 illustrations. The aim should be the presentation of the subject clearly and concisely, avoiding digressions and redundance. Manuscripts are to be written in the third person. Copies of an article entitled "Hints on the Writing of Technical Articles" can be obtained from the Editor of *The Draughtsman*.

Drawings for illustrations should be done either on a good plain white paper or tracing cloth, deep black Indian ink being used. For ordinary purposes they should be made about one-and-a-half times the intended finished size, and it should be arranged that wherever possible these shall not be greater than a single full page of the pamphlet, as folded pages are objectionable, although, upon occasion, unavoidable. Where drawings are made larger, involving a greater reduction, the lines should be made slightly heavier and the printing rather larger than normal, as the greater reduction tends to make the lines appear faint and the printing excessively small in the reproduction. In the case of charts or curves set out on squared paper, either all the squares should be inked in, or the chart or curve should be retraced and the requisite squares inked in. Figures should be as self-evident as possible. Data should be presented in graphical form. Extensive tabular matter, if unavoidable, should be made into appendices.

Authors of pamphlets are requested to adhere to the standard symbols of the British Standards Institution, where lists of such standard symbols have been issued, as in the case of the electrical and other industries, and also to the *British Standard Engineering Symbols and Abbreviations*, No. 1991, published by the B.S.I. at 6/-. Attention might also be given to mathematical notation, where alternative methods exist, to ensure the minimum trouble in setting up by the printer.

The value of the pamphlet will be enhanced by stating where further information on the subject can be obtained. This should be given in the form of footnotes or a bibliography, including the name and initials of the author, title, publisher, and year of publication. When periodicals are referred to, volume and page also should be given. References should be checked carefully.

Manuscripts, in the first instance should be submitted to the Editor, *The Draughtsman*, Onslow Hall, Little Green, Richmond, Surrey.

For pamphlets, a grant of £30 will be made to the author, but special consideration will be given in the case of much larger pamphlets which may involve more than the usual amount of preparation.

Milne, Tannahill & Methven
Printers Ltd
12-14 Mill Street Perth